EFFECTS OF VEHICLE TYPE ON HIGHWAY TRAFFIC FLOW

Effects of vehicle type on speed, delay and capacity characteristics of highway traffic flow in the United Kingdom and Saudi Arabia determined by an examination of traffic data

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by

Z1054831

ALI AL-AKHDAR ALKAIM

(B.Sc.)

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ABSTRACT

The thesis considers the effects of vehicle type on highway traffic flow. The effects of vehicle type on the capacity of traffic signal approaches are examined by the experimental determination of passenger car units at intersections in London and West Yorkshire and in addition saturation flows and lost times are examined. Vehicle type effects at roundabout entries are investigated and the results of field observations reported. Details are given of the gap acceptance of varying vehicle types, the effect of vehicle type on delay and comparisons are made with existing recommendations for the capacity design of roundabout entries. Observations of traffic flow on a rural motorway are used to demonstrate the effect of vehicle type on speed and observed values are fitted to a normal distribution. Overtaking behaviour is also examined and conclusions drawn of the relative effect on capacity of vehicle type. A review is given of the effects of vehicle type on the design and operation of the highway system in Saudi Arabia.

1

Introduction

CHAPTER ONE

Introduction

1.1 - Vehicle features

1.1.1 - Size

The problem of the basic philosophy of vehicle size was initially recognized as early as 1920 in order to solve the productivity problem. Any increase in the allowable vehicle size would bring with it some gains in the economical use of highway transport, but this may cause highway deterioration, wider and higher load-rated bridges, decreasing visibility to other road users, to some extent diminishing trucks' operational ability as well as providing an economic advantage over other modes of transportation.

Truck width, height and length exert an influence on freight-loading practices, terminal facilities and the inter-city operation. They affect pavement and shoulder width, overhead clearance of structures, traffic safety and the movement of passenger cars. So the dimensional and performance characteristics of vehicles are basic to the regulation of road use and to the design of routes and terminals.

In the traffic stream of today, there are three main categories of vehicles, private passenger cars, light and heavy goods vehicles and buses. In each category there is a wide range of makes and types of different performances. Different percentages of these categories present in the traffic stream necessitate the adoption of a suitable design standard.

Highway design, quality of traffic movement, and both vehicle and roadway cost are affected directly by the dimensional aspects of vehicles. Pavement thickness, lane width, terminal layout, sight distance, and vertical clearance are some of the design features affected by the dimensional characteristics of vehicles. Also they influence the vehicle operating costs because of the fashion in which certain resistance forces (air, rolling inertia, etc.) are related to the vehicle design (e.g. fronted area). However, width of vehicle naturally affects the width of traffic lane, length has a bearing on roadway capacity and affects the turning radius and height of vehicle affects clearance of the various structures. Car sizes are of particular significance in connection with modern automobile usage particularly in terms of traffic congestion and design of parking facilities.

Traffic data are required at national, regional and local levels. Without a reliable data base, forward planning has to rely on judgements taken without sufficient information. In Great Britain as in most other countries, regulations which limit the maximum size of any vehicle used on public roads are given in the Motor

Vehicles (Construction and Use) Regulation. Table (1.1.1-1) gives a summary of the maximum legal dimensions and weight of British commercial vehicles (1).

In a report by the Transport and Road Research Laboratory (2), it was suggested that in calculating the likely number of heavier vehicles it was assumed that the maximum length of articulated vehicles will be increased from 15m to 15.5m with a maximum trailer length of 12.3m and that the limit for draw-bar combinations will remain at 18m in line with EEC Draft Directive 71/27/EEC. The 1977 and 1978 data show that about half the sample had a platform length of 12m or more, one quarter were between 10 and 12m and one quarter between 7 and 10m in length. The average for the sample is approximately 11.2m from which it is estimated that the average overall vehicle length is approximately 14.2m, assuming a dimension of 3.2m from the front of articulated vehicle to the front of its trailer.

Bennett (3) has investigated the physical characteristics of the whole motor vehicle fleet of Great Britain. The distribution of the characteristics have been studied by considering the characteristics of various models and weighting these by the frequency of

Type of Vehicle	Wheel Arrangement	Maximum permitted dimension and weight				Note
		Laden Weight	Length	Width	Height	
		Tons	ft (m)	ft (m)	ft (m)	1 - There are few set limits for vehicles constructed and normally used to carry abnormal indivisible loads. The measurements quoted are the maximum which do not require police notification of journeys.
Rigid Goods	2 axles	16	36'-11" (11)	8'-2½" (2.5)	-	
	3 axles	22				
	4 or more axles	28				
Separate trailer	Less than 6 wheels	22	22'-11½ (7)	8'-2½" (2.5)	-	
Articulated Goods	3 axels	22	43'-7¾" (13.31)	8'-2½" (2.5)	-	
	4 axles	32				
Low loaders	-	75	50 (15.25)	9'-6" (2.9)		
P.S.V.	-	16	36 (10.98)	8'-2½" (2.5)	15' (4.58)	

Table 1.1.1-1 Maximum legal dimensions and weight of British commercial vehicles.
(Reproduced from reference No.1)

each model on the road system. The mean values of the sizes of different types of vehicles are shown in Table (1.1.1-2). Table (1.1.1-3) shows a comparison of maximum legal dimensions and weights for goods vehicles in European countries, Australia and the U.S.A.

Recent data for the top twenty car models (6), (9) of new registrations in the United Kingdom are shown in Tables (1.1.1-4 to 1.1.1-7) and the mean values of overall length, width and height of passenger cars are plotted as shown in Figures (1.1.1-1 to 1.1.1-3) which covers four years' models starting 1981 (6), (9).

The data were analysed to obtain the mean values for overall length, width and height of different types of passenger cars. The results show a slight increase in passenger car dimensions and a slight variation in car sizes when compared with results found by Bennett.

Type of Vehicle	Vehicle Dimensions Mean Value (m)		
	Length	Width	Height
Cars and Car Derived Vans	4.1	1.61	1.38
Single Unit Commercial Vehicles Excluding Car Derived Vans	5.48	2.05	3.31
Single Unit Vehicles of over 3.5 tons Rated Maximum Weight	7.04	2.29	4.19

Table 1.1.1.-2 Vehicle dimensions mean values for the period covering 1961 to 1974.
(Reproduced from reference No.3)

Country	Maximum legal dimensions and weight 1978				
	Width	Length		Laden Weight	
	ft(m)	Rigid ft(m)	Artic. ft(m)	Rigid Tons	Artic. Tons
G.B.	8' 0" (2.43)	30' 0" (9.10)	35' 0" (10.62)	24	24
France	8' 2.5" (2.50)	36' 1" (10.95)	45' 11¼" (13.93)	26	26
W. Germany	8' 25" (2.50)	36' 1" (10.95)	49' 2½" (14.93)	22	32
Italy	8' 2.5" (2.50)	36' 1" (10.95)	45' 11¼" (13.93)	21.7	31.5
Australia*-1	8' 0" (2.43)	31' 0" (9.40)	45' 0" (13.65)	*-2	*-2
Canada*-1	8' 0" (2.43)	36' 0" (10.62)	50' to 60' (15.2 - 18.20)	*-2	*-2
U.S.A.*-1	8' 0" (2.43)	35' 0" (10.62)	50' to 60' (15.2 - 18.20)	18 to 24	28 to 33
G.B. (Proposed Revision)	8' 2.5" (2.50)	36' 1" (10.95)	42' 7¾" (12.94)	28	32

Table 1.1.1-3 Max. legal dimension and weight in different countries (goods vehicles). (Reproduced from reference No.19)

*1 - Legal requirements vary for individual states or province.

*2 - Weight regulations are too complex to be summarized.

Type of Passenger Car	No.	%	Overall Length (m)	Width (m)	Height (m)
1 Ford Cortina	159,804	10.76	4.33	1.70	1.35
2 Ford Escort	141,081	9.50	3.96	1.65	1.37
3 Ford Fiesta	110,753	7.46	3.66	1.58	1.32
4 Austin/MG Metro	110,283	7.43	3.41	1.55	1.37
5 Morris Ital	48,490	3.27	4.35	1.64	1.39
6 Vauxhall Chevette	36,838	2.48	4.17	1.58	1.32
7 Vauxhall Cavalier	33,631	2.27	4.37	1.68	1.40
8 Nissan Cherry	32,874	2.21	3.96	1.63	1.40
9 Vauxhall Astra	30,854	2.08	3.99	1.63	1.37
10 Mini	28,772	1.94	3.05	1.42	1.32
11 VW Golf	26,413	1.78	3.99	1.67	1.42
12 Nissan Sunny	25,737	1.73	4.27	1.37	1.37
13 Renault 5	25,220	1.70	3.53	1.52	1.41
14 Ford Granada	25,214	1.70	4.78	1.80	1.42
15 Volvo 300 Series	23,775	1.60	4.30	1.66	1.39
16 Ford Capri	22,289	1.50	4.37	1.70	1.35
17 Rover	21,504	1.45	4.75	1.78	1.40
18 Renault 18	21,127	1.42	4.39	1.69	1.40
19 Volvo 200 Series	20,778	1.40	4.79	1.71	1.46
20 A/M Allegro	20,753	1.40	3.91	1.61	1.37
	Total = 1,484,713				
Mean value			(4.02)	(1.63)	(1.37)

Table 1.1.1-4 Top twenty passenger cars in the U.K. (1981). (From references 6 and 9)

Type of Passenger Car	No.	%	Overall Length (m)	Width (m)	Height (m)
1 Ford Escort	166,942	10.74	3.96	1.65	1.37
2 Ford Cortina	135,745	8.73	4.33	1.70	1.35
3 Austin/MG Metro	114,550	7.37	3.41	1.55	1.37
4 Ford Fiesta	110,165	7.08	3.66	1.58	1.32
5 Vauxhall Cavalier	100,081	6.44	4.37	1.68	1.40
6 Vauxhall Astra	46,412	2.98	3.99	1.63	1.37
7 Triumph Acclaim	42,188	2.71	4.09	1.60	1.35
8 Volvo 300 Series	30,412	1.96	4.30	1.66	1.39
9 Nissan Sunny	28,744	1.85	4.27	1.37	1.37
10 Ford Granada	28,590	1.84	4.46	1.73	1.45
11 Nissan Cherry	27,711	1.78	3.96	1.63	1.40
12 VW Golf	26,311	1.69	3.99	1.67	1.42
13 Mini	25,503	1.64	3.05	1.42	1.32
14 Austin Ambassador	24,678	1.59	4.46	1.73	1.45
15 Rover	24,420	1.57	4.75	1.78	1.40
16 Vauxhall Chevette	23,842	1.53	4.17	1.58	1.32
17 VW polo	23,763	1.53	3.66	1.58	1.36
18 Morris Ital	23,228	1.49	4.35	1.64	1.39
19 Volvo 200 Series	20,518	1.32	4.79	1.71	1.46
20 Nissan Stanza	20,030	1.29	3.65	1.66	1.41

Total = 1,555,027
Mean value (4.02)(1.63) (1.37)

Table 1.1.1-5 Top twenty passenger cars in the U.K. (1982). (From references 6 and 9)

Type of Passenger Car	No.	%	Overall Length (m)	Width (m)	Height (m)
1 Ford Escort	174,190	9.72	3.96	1.65	1.37
2 Ford Sierra	159,119	8.88	4.52	1.73	1.45
3 Austin/MG Metro	137,303	7.66	3.41	1.55	1.37
4 Vauxhall Cavalier	127,509	7.12	4.37	1.68	1.40
5 Ford Fiesta	119,602	6.68	3.66	1.58	1.32
6 Austin/MG Maestro	65,328	3.65	4.04	1.68	1.42
7 Vauxhall Astra	62,570	3.49	3.99	1.63	1.37
8 Triumph Acclaim	38,406	2.14	4.09	1.60	1.35
9 Nissan Sunny	36,781	2.05	4.27	1.37	1.37
10 Volvo 300 Series	36,753	2.05	4.30	1.66	1.39
11 VW Polo	32,706	1.83	3.66	1.58	1.36
12 Nissan Cherry	29,229	1.63	3.96	1.63	1.40
13 Mini	27,739	1.55	3.05	1.42	1.32
14 VW Golf	25,764	1.44	3.99	1.67	1.42
15 Vauxhall Nova	24,995	1.40	3.96	1.70	1.35
16 Ford Granada	24,076	1.34	4.68	1.80	1.42
17 Ford Capri	22,254	1.24	4.37	1.70	1.35
18 Rover	21,587	1.20	4.75	1.78	1.40
19 Volvo 200 Series	21,381	1.19	4.79	1.71	1.46
20 Talbot Samba	20,105	1.12	3.51	1.53	1.37
	Total = 1,791,669				
	Mean value		(4.03)	(1.64)	(1.38)

Table 1.1.1-6 Top twenty passenger cars in the U.K. (1983). (From references 6 and 9)

Type of Passenger Car	No.	%	Over-all Length (m)	Width (m)	Height (m)
1 Ford Escort	157,340	8.99	3.96	1.65	1.37
2 Vauxhall Cavalier	132,149	7.55	4.37	1.68	1.40
3 Ford Fiesta	125,851	7.19	3.66	1.58	1.32
4 Austin/MG Metro	117,442	6.71	4.04	1.68	1.42
5 Ford Sierra	113,071	6.46	4.52	1.73	1.45
6 Austin/MG Maestro	83,072	4.75	4.04	1.68	1.42
7 Vauxhall Astra	56,511	3.23	3.99	1.63	1.37
8 Vauxhall Nova	55,442	3.17	3.96	1.70	1.35
9 Ford Orion	51,026	2.92	3.96	1.70	1.37
10 Volvo 300 Series	35,034	2.00	4.30	1.66	1.39
11 Austin/MG Montego	34,728	1.98	4.04	1.68	1.42
12 VW Polo	31,345	1.79	3.66	1.58	1.36
13 Nissan Sunny	31,112	1.78	4.27	1.37	1.37
14 Nissan Cherry	27,466	1.57	3.96	1.63	1.40
15 VW Golf	24,250	1.39	3.99	1.67	1.42
16 Mini	23,329	1.33	3.05	1.42	1.32
17 Ford Granada	23,215	1.33	4.78	1.80	1.42
18 Fiat Uno	20,915	1.20	3.66	1.55	1.42
19 Nissan Micra	20,789	1.19	3.65	1.56	1.40
20 Renault 11	20,394	1.17	3.98	1.66	1.41

Total = 1,749,650
Mean value (4.04) (1.65) (1.39)

Table 1.1.1-7 Top twenty passenger cars in the U.K. (1984).
(From references 6 and 9)

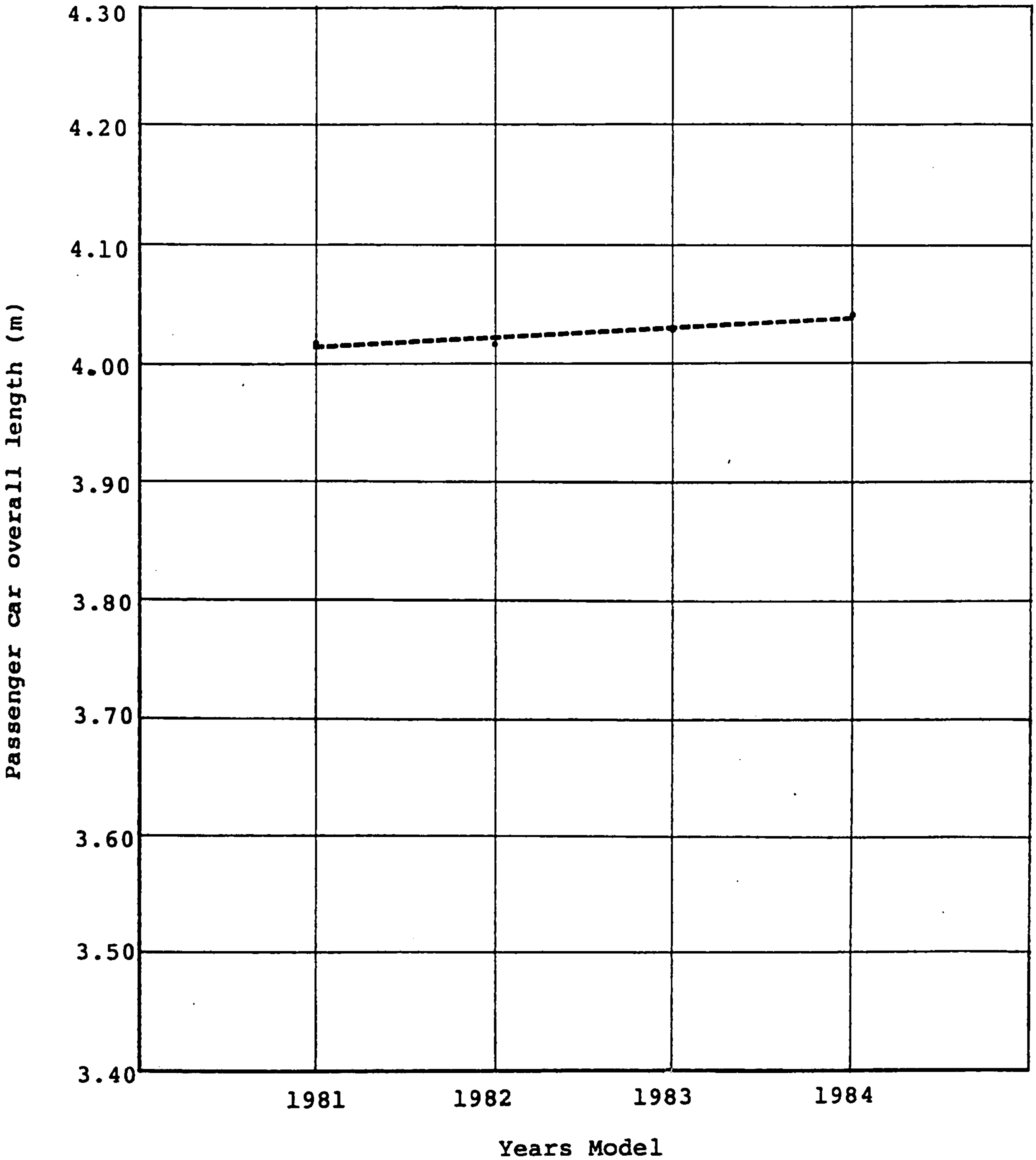

Figure 1.1.1-1 Average overall length of top twenty passenger cars in the U.K.

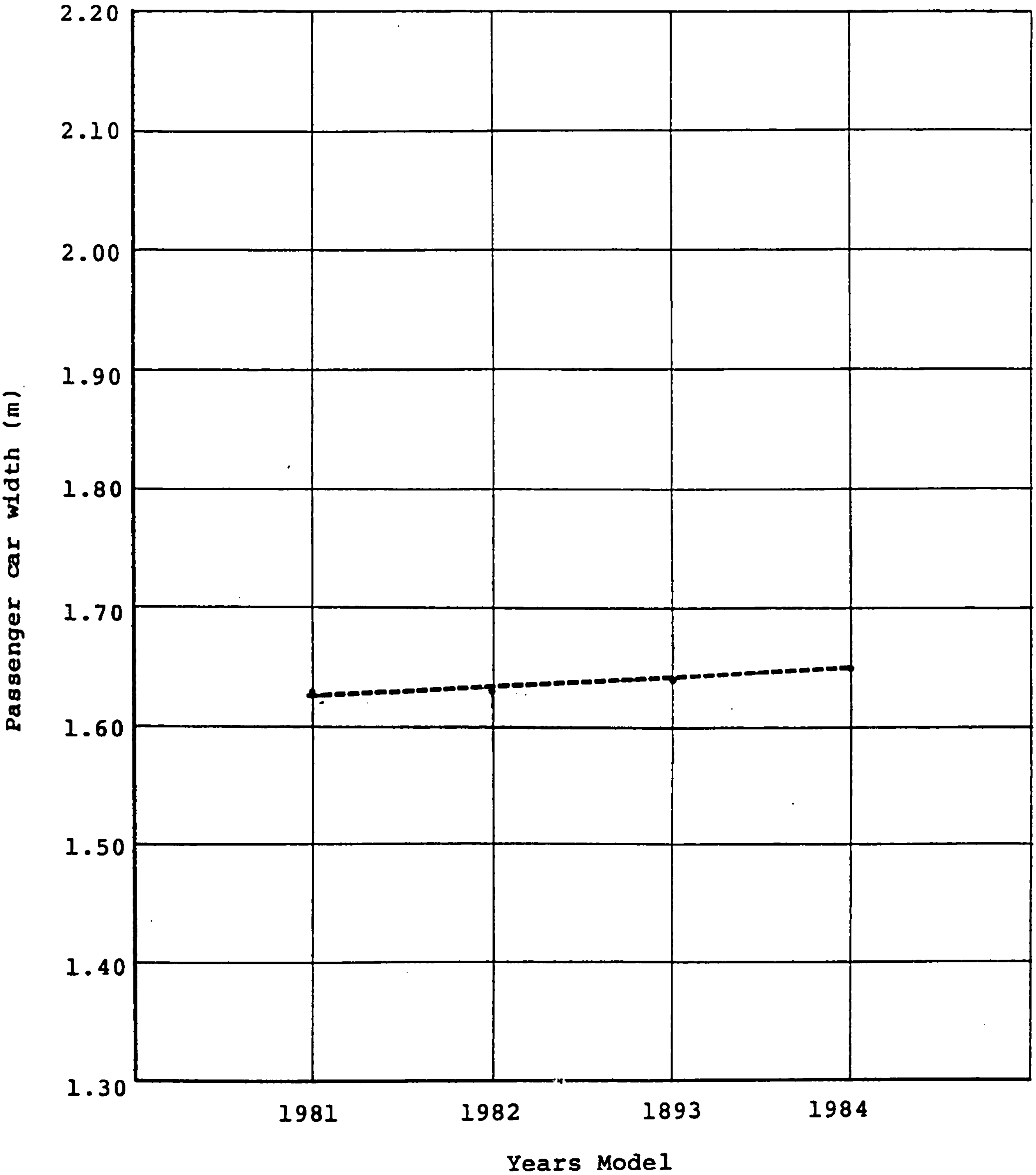

Figure 1.1.1-2 Average width of top twenty passenger cars in the U.K.

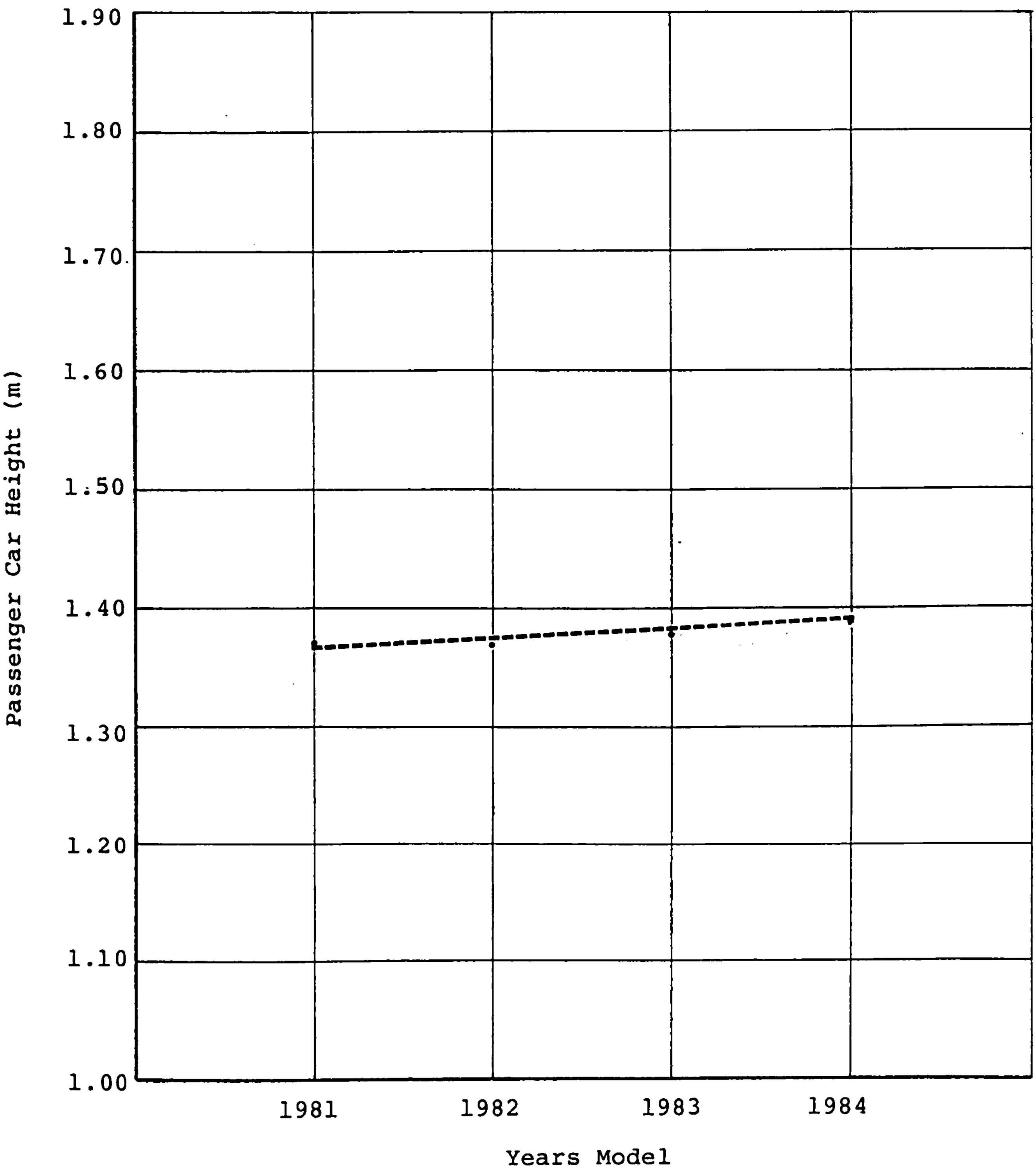

Figure 1.1.1-3 Average height of top twenty passenger cars in the U.K.

1.1.2 - Weight

The weight of a vehicle is usually expressed in terms of its axle load which is the total load transmitted to the road. Axle loading is an important consideration in the design of roads; the standard axle load is a useful tool in the structural design of pavements.

In the early nineteen-fifties, the first measurements of wheel load (i.e. the load on one end of an axle) in Great Britain were made by the Road Research Laboratory (4) in two full-scale road experiments, on A1 north of Boroughbridge in Yorkshire and on A38 at Cambridge in Gloucestershire. A sample of the commercial traffic was stopped and weighed using simple manually operated weighing platforms. At a later date, attention was focused on the need to develop an automatic method of weighing commercial traffic in motion, using weighbridges built into the road structure.

At the Alconbury Hill experiment on Trunk Road A1, the first automatic unit was installed by the Laboratory in 1958. The equipment has been further developed since that time, both in the modules of the weighing platform and in the recording and analysis of the wheel load data (4). The system is to be improved further to widen the scope of the data obtained to include information on axle spacing that will enable articulated and rigid vehicles to be distinguished.

In 1964 a number of weighbridges were installed at Alconbury By-Pass (A1), Wheatley By-Pass (A40), Nately Scures (A30), Whitfield (A38) and Tamworth (A4091) by the Laboratory and in 1969 systematic measurements were possible at these sites. In addition to this, the installation of some weighbridges was carried out on M1, M6 and M4,and in 1976 two weighbridges were installed to measure the commercial traffic passing over a full-scale road experiment on the access road to the Grangemouth refinery and docks (4).

The weight of commercial vehicles in Britain is governed by the Motor Vehicle (Construction and Use) Regulations. These regulations permit vehicles conforming to them to move freely over the road system apart from some special restrictions applying generally to bridges. The maximum axle loading of specially authorised vehicles of greater dimensions is 22 tons. Results of observed overweight vehicles by the Transport and Road Research Laboratory (5) showed that the greatest number of overweight axles were on 2-axle vehicles.

It was suggested that this would be expected because of the normally high percentage of such commercial vehicles. Table (1.1.2-1) was introduced by the Transport and Road Research Laboratory (10) where the proportions of six types of commercial vehicles were known for a particular road and it provides a method of calculating the average number of standard axles per commercial vehicle.

Bennett 1976 (3) investigated the frequency distribution of motor vehicles' weight which included the distribution of maximum-rated weights for all vehicles (including articulated vehicles and buses) rated over 3.5 tons gross. The results are shown in Figure (1.1.2-1) which shows that 98 per cent have 3.6 tons weight full and 2 per cent have 30.5 tons with a mean value of 13.9 tons.

The Transport and Road Research Laboratory 1981 (5) suggested that the number of commercial vehicles has altered little since the early sixties but there has been a considerable increase in allowable gross weights and a dramatic increase in the use of the heaviest 4 and 5 axle articulated vehicles. A comparison between the situation in 1962 and 1978 is shown in Table (1.1.2-2).

Vehicle Type	% present	1st axle	2nd axle	3rd axle	4th axle	5th axle	Total
2 axle rigid	51%	3.63	27.02				30.65
3 axle articulated	15%	0.59	8.47	9.09			18.14
3 axle rigid	7%	1.68	7.27	5.07			14.02
4 axle articulated	20%	2.06	30.10	12.23	13.56		57.96
4 axle rigid	5%	0.42	0.31	6.36	4.99		12.08
5 axle articulated	2%	0.19	2.77	1.25	1.14	1.08	6.43
Total							139.28
139.28/100 equals 1.39 standard axles per commercial vehicle							

Table 1.1.2-1 Number of standard axles for each axle (multiply per cent present by number of standard axles/100.) (Reproduced from reference No.10)

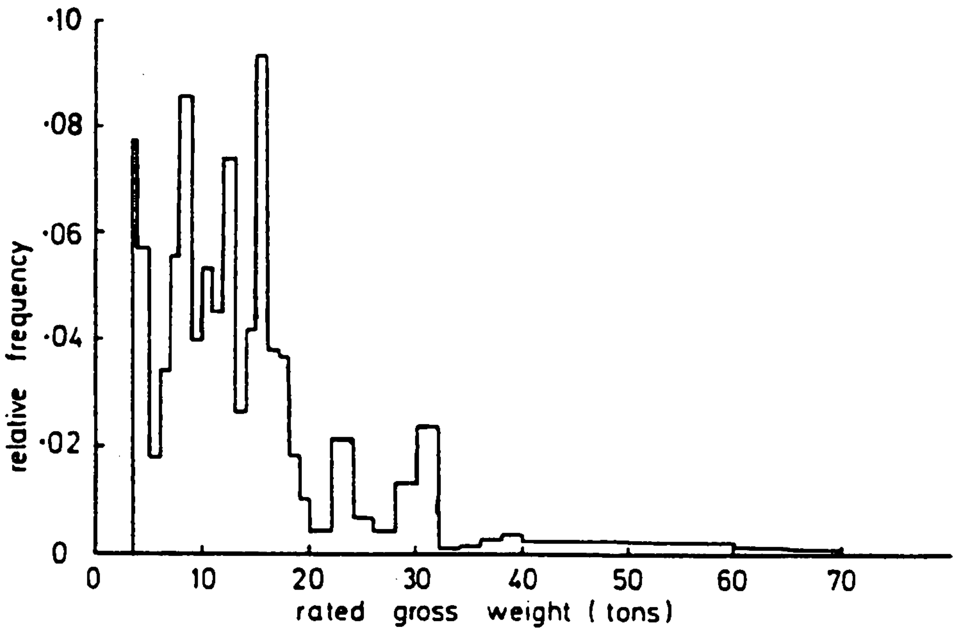

Figure 1.1.2-1 Distribution of rated gross weights for all vehicles rated at over 3.5 tons registered in Great Britain between 1961 and 1974. (Reproduced from reference No.3)

	1962		1978	
Road System Length (thousand km) Total	316		337	
Motorway	0.2		2.4	
	Rigid	Artic.	Rigid	Artic.
Max. gross wt. (ton)	14	24	16	32
Max. length (m)	9	11	11	15
Number (thousands)	440	55	332	104

Table 1.1.2-2 A comparison of commercial vehicles between 1962 - 1978 in the U.K. (Reproduced from reference No.4)

The 25 tonnes gross weight vehicles are predicted to grow at a faster rate than commercial vehicles of lower weight.

With regard to the economy aspect, there should be some balance in the restrictions imposed both on trucks and on roads, so that the net of compensation for the wear on the road and the cost of truck tonnage transportation is reduced to its minimum. This could be achieved by adopting more realistic and more modern standards.

In a report by the Transport and Road Research Laboratory, 1980 (2) operating costs and road damage factors were investigated by using a range of nine types of heavy goods vehicles. The maximum axle weight was restricted for six of the designs to 10 tonnes and in 2 of the designs to 11 tonnes drive axles.

In this report the design parameters for a complete assessment of the results were:-

i) Number of axles;
ii) Maximum gross vehicle weight;
iii) Unladen weight;
iv) Payload weight capacity;
v) Available length and volume for payload;

vi) Axle weight distribution;

vii) King-pin load;

viii) Traction ratio;

ix) Allowable longitudinal movement in the centre of gravity of the payload without violating axle weight or traction ratio limits;

x) Turning circle radii;

xi) Rear overhang;

xii) Power output of engine and fuel consumption.

The conclusions of the report stated that the total number of vehicles of 32.5 tonnes gross vehicle weight or more will be reduced by about 8% within a few years of the change in permitted weight to 34 tonnes, and by a further 5% after 5 to 10 years.

In comparison with 1977 levels, the road damage due to vehicles of 32.5 tonnes gross vehicle weight or more was expected to be reduced if axle weights were limited to 10 tonnes overall. The reduction was stated to be between 7% and 18% depending on the type of vehicle permitted. If the vehicles with 11 tonne drive axles proposed in the EEC Draft Directive are permitted then overall road damage is expected to be reduced compared to 1977 levels.

1.1.3 - Types (Classification)

Vehicles are classified according to their physical features and their operating characteristics. This classification is necessary for highway design because of the relevant information that is useful both in geometric layout and traffic control. In the inner cities the passenger car, because of its higher percentages present in the traffic system, and the provision of parking, represents the most important factor which designers need. The articulated goods vehicle is an essential factor in determining the queue length at traffic control points, the storage length of lane, turning radius and corner radii at intersections.

On the other hand the percentages of the various vehicle types present in the traffic stream dictate the adoption of a suitable design for the pavement thickness, lane width, terminal layout, sight distance and vertical clearance.

A limited classification, which has been of great use on a wide variety of roads in the United Kingdom (11) is as follows:-

Classification	Includes
Passenger car	Motor cycle combination; motor cars; light goods vehicles (<1500 Kg unladen)
Commercial vehicles	Goods vehicles (>1500 Kg unladen)
Buses	All passenger service vehicles.

Figure (1.1.3-1) shows the wide range of types of vehicles found in the heterogeneous traffic stream today. The vehicle class listing was introduced by the Transport and Road Research Laboratory (2) which is compatible with EEC regulation R1108/70.

The 1935 to 1984 figures for vehicles in use in the United Kingdom (6) are shown in Table (1.1.3-1), and the 1984 vehicles in use are classified into percentages in Figure (1.1.3-2). The figure shows that cars are more numerous than all other classes of vehicle together, and this is expected to be even more so in the future. This is shown in Table (1.1.3-2) where the forecasts of vehicles in use were taken from Basic Road Statistics for 1983 for the period from 1980 to 2010 (7).

Class No	Vehicle description
0	Moped, scooter motorcycle
1	Car, light van, taxi
2	Light goods vehicle
21	Car or light goods vehicle + 1 axle caravan or trailer
22	Car or light goods vehicle + 2 axle caravan or trailer
31	Rigid 2 axle heavy goods vehicle
32	Rigid 3 axle heavy goods vehicle
33	Rigid 4 axle heavy goods vehicle
34	Rigid 3 axle heavy goods vehicle
35	Rigid 4 axle heavy goods vehicle
41	Rigid 2 axle HGV + 2 axle drawbar trailer
42	Rigid 2 axle HGV + 3 axle drawbar trailer
43	Rigid 3 axle HGV + 2 axle drawbar trailer
44	Rigid 3 axle HGV + 3 axle drawbar trailer
45	Rigid 2 axle HGV + 1 axle caravan or trailer
46	Rigid 2 axle HGV + 2 axle (close coupled) trailer
51	Artic, 2 axle tractor + 1 axle semi-trailer
52	Artic, 2 axle tractor + 2 axle semi-trailer
53	Artic, 3 axle tractor + 1 axle semi-trailer
54	Artic, 3 axle tractor + 2 axle semi-trailer
55	Artic, 2 axle tractor + 3 axle semi–trailer
56	Artic, 3 axle tractor + 3 axle semi-trailer
61	Bus or coach, 2 axle
62	Bus or coach, 3 axle
7	Vehicle with 7 or more axles
2N	2 axle vehicle not otherwise classified
3N	3 axle vehicle not otherwise classified
4N	4 axle vehicle not otherwise classified
5N	5 axle vehicle not otherwise classified
6N	6 axle vehicle not otherwise classified

Figure 1.1.3-1 TRRL VEHICLE CLASS LISTING COMPATIBLE WITH EEC REGULATION R1108/70

(Reproduced from reference No.2)

SUMMARY OF MOTOR VEHICLES IN USE

.IN THE UNITED KINGDOM*

		Private & Light Goods	Taxis Buses & Coaches†	Heavy Goods	Tractors‡	Exempt Vehicles	Agric. Tractors (etc) £15.00 Class	Motor Cycles§	TOTAL
Sept.:	1935	1,505,019	87,383	442,187	3,069	33,057	19,407	521,128	2,611,250
	1936	1,675,104	87,820	467,561	2,820	38,562	22,610	510,242	2,804,719
	1937	1,834,248	87,474	487,750	2,915	49,446	25,704	491,718	2,979,255
	1938	1,984,430	89,410	504,028	2,373	63,336	29,554	466,265	3,139,396
Aug. 31st	1939	2,073,404	91,747	496,927	2,884	84,890	37,361	421,205	3,208,418
	1940	1,456,144	82,982	452,478	2,576	47,587	49,800	281,367	2,372,934
	1941	1,538,060	86,871	459,684	3,405	55,368	66,100	320,468	2,529,956
	1942	880,207	86,922	463,355	4,286	46,857	90,100	309,381	1,881,108
	1943	735,248	88,732	459,682	4,140	52,012	106,300	124,802	1,570,916
	1944	773,034	92,723	458,178	3,498	55,727	117,700	125,013	1,625,873
	1945	1,521,581	101,003	483,867	3,660	57,442	125,800	312,844	2,606,197
Sept.:	1946	1,807,067	107,025	572,432	3,370	61,593	155,471	467,058	3,174,016
	1947	1,983,505	117,021	684,102	3,811	72,690	194,533	533,783	3,589,445
	1948	2,002,201	130,519	785,334	4,123	79,823	242,906	560,107	3,805,013
	1949	2,178,411	136,892	862,136	4,238	75,383	283,836	655,200	4,196,096
	1950	2,307,379	139,609	915,566	4,335	68,930	313,992	761,500	4,511,311
	1951	2,433,172	138,670	955,003	4,384	71,283¶	301,360	859,034	4,762,906
	1952	2,564,686	135,261	985,832	4,687	97,925	324,132	962,210	5,074,733
	1953	2,824,789	118,583	1,018,453	4,882	100,822	347,811	1,052,864	5,468,204
	1954	3,172,869	110,186	1,056,391	5,053	100,977	367,403	1,156,568	5,969,447
	1955	3,609,400	104,777	1,134,257	5,453	102,105	390,962	1,276,894	6,623,848
	1956	3,980,511	101,692	1,200,588	5,711	107,949	403,172	1,349,282	7,148,905
	1957	4,282,438	99,398	1,244,533	5,842	110,277	424,968	1,497,050	7,664,506
	1958	4,651,021	97,761	1,298,320	5,794	110,755	439,669	1,546,183	8,149,503
	1959	5,080,510	94,282	1,358,855	6,197	111,209	459,370	1,764,535	8,874,958
	1960	5,650,461	95,302	1,432,475	6,376	116,303	471,192	1,894,372	9,666,481
	1961ø	6,113,764	93,512	1,490,007	6,535	121,369	481,420	1,920,832	10,227,439
	1962ø	6,706,159	94,896	1,511,060	5,967	123,848	483,312	1,897,440	10,822,682
	1963ø	7,546,650	98,173	1,571,772	6,206	131,745	496,401	1,878,312	11,729,259
	1964ø	8,436,193	98,644	1,619,644	6,836	137,838	506,795	1,866,128	12,672,199
	1965ø	9,131,075	98,443	1,643,353	6,824	145,251	500,719	1,735,495	13,261,160
	1966ø	9,746,887	95,895	1,610,446	6,757	162,639	478,335	1,519,981	13,620,940
	1967ø	10,554,193	96,350	1,661,854	6,644	168,305	496,578	1,463,764	14,447,688
	1968ø	11,078,000	101,500	1,606,800	6,400	179,700	489,300	1,343,000	14,804,700
	1969ø	11,504,300	104,060	1,604,690	5,880	186,410	475,510	1,238,920	15,119,770
	1970ø	11,801,777	105,421	1,658,261	5,697	138,556	457,392	1,155,440	15,322,544
	1971ø	12,357,868	108,669	1,660,223	5,762	144,157	449,802	1,033,150	15,759,631
	1972ø	13,022,764	106,957	1,686,158	5,962	147,416	436,660	1,075,770	16,481,687
	1973ø	13,815,000	108,880	1,766,430	4,750	156,980	438,640	1,122,820	17,413,500
	1974ø	13,947,934	108,884	1,800,650	5,696	177,165	441,001	1,154,461	17,635,791
	1975ø	14,060,973	113,694	1,813,075	6,977	183,381	424,889	1,281,576	17,884,565
	1976ø	14,372,834	115,392	1,796,281	7,211	180,775	414,365	1,347,100	18,233,958
	1977●	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
	1978$	14,416,989	112,206	1,742,528	7,475	254,343	407,530	1,314,816	18,255,887
Dec.:	1979	14,926,571	112,781	1,820,051	7,555	381,010	412,654	1,405,435	19,066,057
	1980	15,437,733	112,502	1,800,335	7,176	408,732	406,597	1,479,505	19,652,580
	1981	15,632,683	112,234	1,770,489	6,888	423,446	372,624	1,473,811	19,792,175
	1982¿	16,074,735	113,290	1,736,211	6,541	451,258	380,284	1,470,070	20,232,389
	1983	16,611,529	115,445	1,575,424	6,165	619,286	385,760	1,384,851	20,698,460
	1984	17,313,409	119,039	1,470,360	5,911	669,366	384,877	1,316,547	21,279,509

* Excludes Channel Islands and Isle of Man.
† Excludes tramcars.
‡ 1935 to 1937 figures contain class of tractor now included with Agric. Tractors for £15.00 class.
§ Includes other licensed vehicles.
¶ Estimated for Great Britain.
ø For Great Britain—sample Census figures only.
● No census was taken in 1977.
$ The 1978 census for Great Britain was taken on the 31st December whereas the Northern Ireland census was at the 30th Sept., also from 1978 the census for Great Britain was a full census and the first taken from the computer file at the D.V.L.C. consequently the figures were compiled by a new method and therefore do not strictly relate to previous years.
¿ From October 1982 all general goods vehicles less than 1525 kgs unladen weight are assessed for vehicle excise duty at the same rate as private vehicles. The above table relates to vehicles *currently licenced by taxation class*

Table 1.1.3.1

Extracted from (Motor Industry of Great Britain 1985)

The Statistical Department

(From reference No 6)

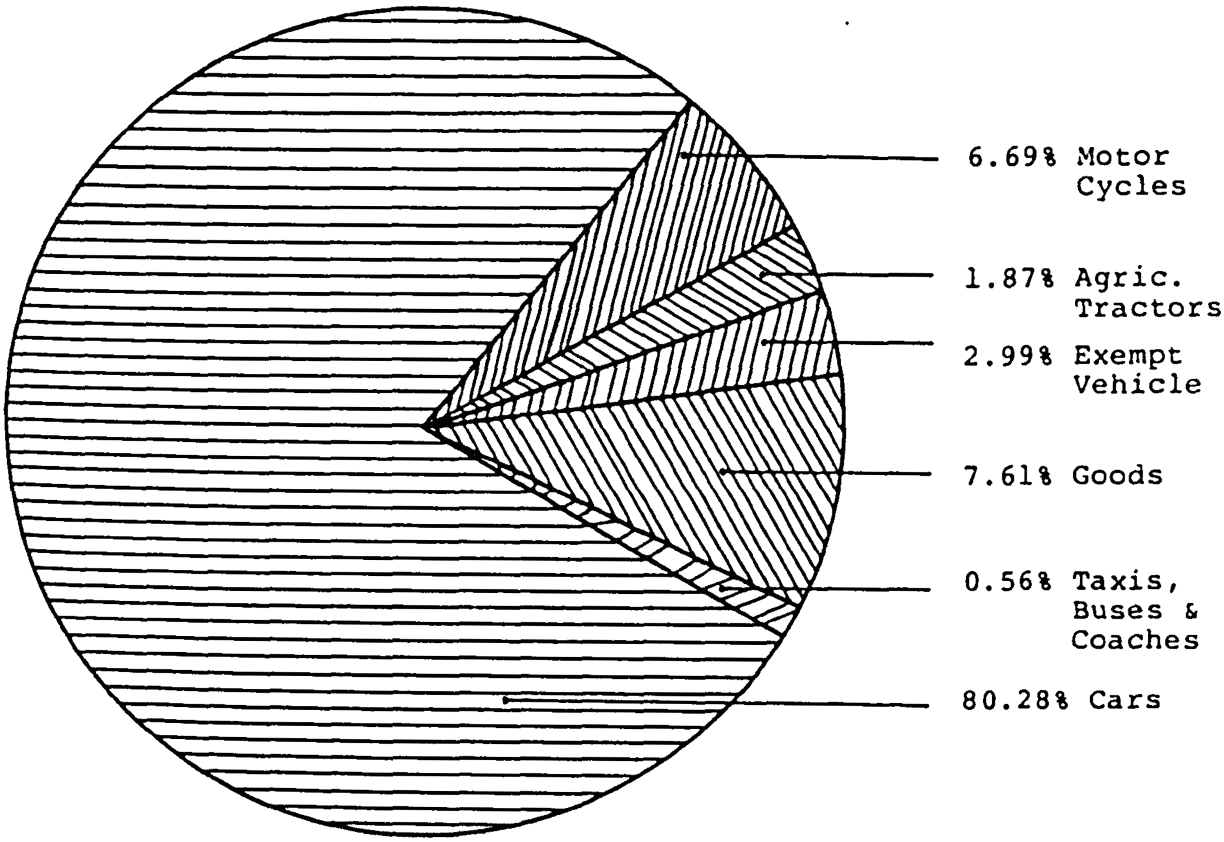

Figure 1.1.3-3

Summary of motor vehicles in use in the United Kingdom (1984).

From the Motor Industry of Great Britain.

Year		Cars	Taxis, buses and coaches	Light vans	Other goods vehicles	Total vehicles*
1980	Actual	15,073	110	1,218	544	18,317
1981	Actual	15,267	109	1,213	524	18,484
1985	*Low*	*16,742*	*110*	*1,337*	*550*	*18,739*
	High	*17,446*		*1,372*	*589*	*19,517*
1990	*Low*	*18,430*	*110*	*1,407*	*539*	*20,486*
	High	*19,837*		*1,487*	*589*	*22,023*
1995	*Low*	*19,837*	*110*	*1,464*	*534*	*21,945*
	High	*21,948*		*1,614*	*594*	*24,266*
2000	*Low*	*20,963*	*110*	*1,533*	*534*	*23,140*
	High	*23,777*		*1,753*	*600*	*26,240*
2005	*Low*	*21,948*	*110*	*1,603*	*528*	*24,189*
	High	*25,184*		*1,914*	*605*	*27,813*
2010	*Low*	*22,792*	*110*	*1,672*	*528*	*25,102*
	High	*26,450*		*2,087*	*605*	*29,252*

* Excluding agricultural tractors, tricycles, pedestrian controlled vehicles, showmen's haulage, crown and exempt vehicles and motorcycles (about 2.25 million vehicles).

Table 1.1.3-2 Forecasts of vehicles in use from Basic Road Statistics up to Year 2010 in Great Britain (1983).
(Reproduced from reference No.7)

Car traffic growth is mainly a result of car ownership growth, as suggested by the Transport and Road Research Laboratory (8) and ownership forecasts are therefore a crucial element in long-term transport planning.

Figures from Table (1.1.3-1) are plotted in Figure (1.1.3-4) which provides the pattern of growth of vehicles in the U.K. It shows the rate of increase in the number of passenger cars for the last fifteen years or so. This rate of increase is also compared with the rate of increase of goods vehicles and the total number of vehicles in use. The results show that passenger cars follow the same pattern as the total number of vehicles in use in their rate of increase, while the goods vehicles'rate of increase is very minute. In general the rate of growth is connected with the country's economy which reflects the need for roads. In 1982 the British economy began to grow again and for that year there was a surprisingly large growth in motor traffic of 7.3% which continued into 1983 with an 8.3% growth in traffic during the first half of the year compared with the first half of 1982.

The Organisation for Economic Co-operation and Development (1983) (12) suggested that during

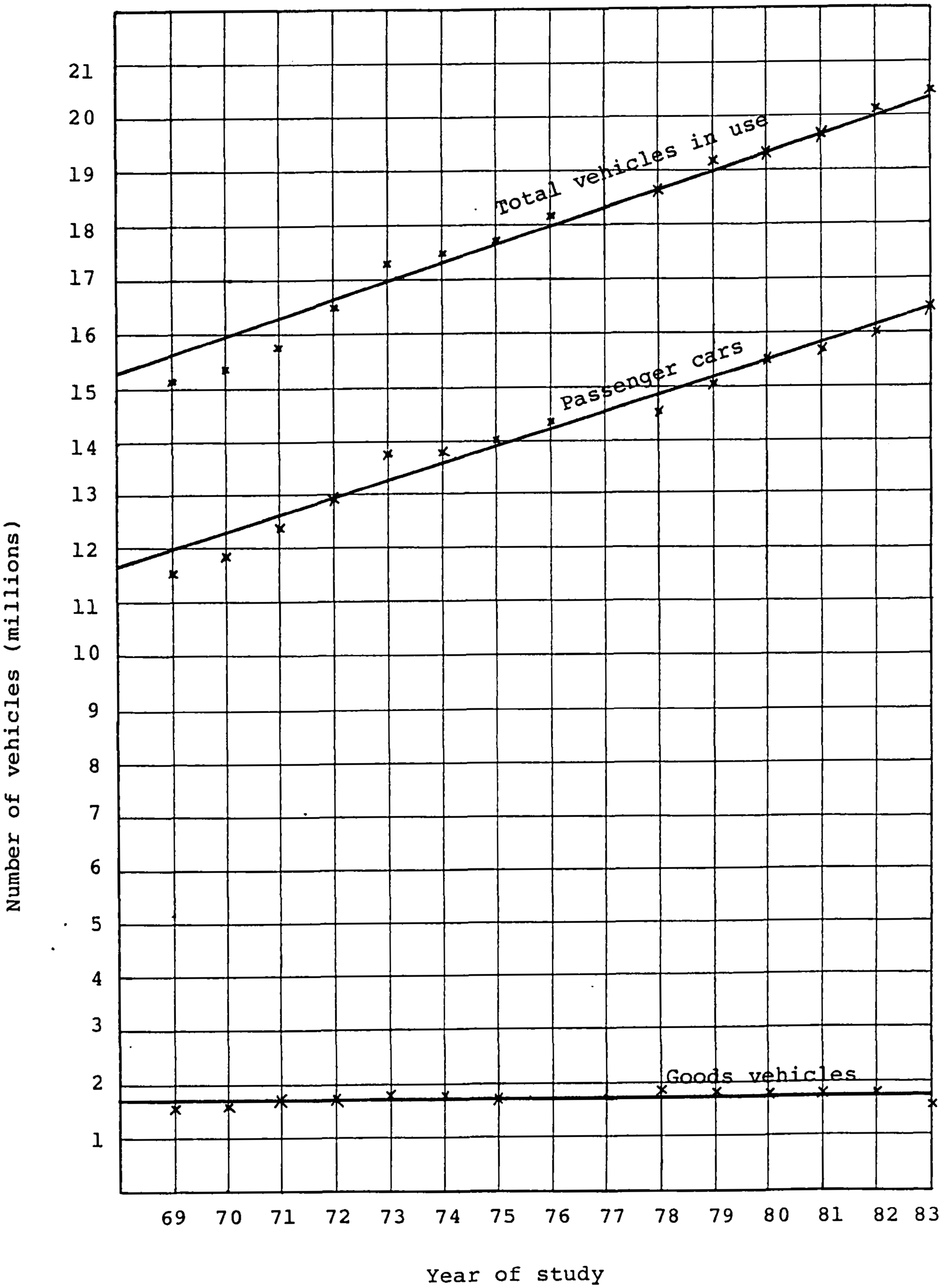

Figure 1.1.3-4 Pattern of growth of vehicles in the U.K.

the period 1985-2000, sales volume is expected to build up to 1.83 million units, indicating a market growth of 0.7 per cent per year. This compares with an annual growth rate of 3.9 per cent in the 1970's. The main factors underlying this marked slow-down will be the successively smaller additions to the vehicle stock as saturation is approached, minimal population growth and a levelling of earlier rises in scrapping rates.

By the year 2000, the total vehicle stock will probably have risen to around 20.7 million units giving a vehicle density of 365/Km.

1.2 - Effect of vehicle type on highway design

1.2.1 - Lane width

The selected design standards for traffic lanes are influenced by the dimensions of motor vehicles. A lane width of 3.65m is considered ideal for heavy volumes of mixed traffic and a narrower lane width will restrict capacity. The design requirement for lane width is naturally governed by the width of the vehicle using it and hence on traffic composition.

Vehicle manoeuvres at intersections are an important consideration, they are divided into the following basic categories:
a) Diverging, b) Merging, c) Weaving, d) Crossing, and the width of the lane which should be provided for vehicles to carry out these manoeuvres are related to vehicle size. The provision of a storage lane for turning vehicles should be of a specified width to coincide with the vehicle type using it in significant numbers so as not to interfere with other vehicle movements using the intersection.

Controlled experiments carried out by the Transport and Road Research Laboratory (13) showed a relationship between the approach width and the value of the number of passenger cars equivalent to goods vehicles. This value was 1.47 for an approach width of 3.65m.

In a report by the Transport and Road Research Laboratory (14), the factors involved in determining the road width required by a vehicle to negotiate horizontal curvature were discussed. The measurements obtained with a sample of commercial vehicles were compared with others obtained using models and by calculation. A sample of 13 vehicles (5 rigid, 7 articulated and one rigid vehicle with draw-bar trailer) were used and the results were:-

a) Articulated vehicles of 15m length and 12m wheel base required a width of 6.5m for a 90" turn, and a 1m increase in wheel base could increase the required road width to over 7m.

b) The rigid vehicle trailer although having combined wheel base and overall length greater than any of the rigid or articulated vehicles, required less road width for a given turn than all but the shortest articulated vehicle and less than the single deck public service vehicle.

c) Maximum size articulated vehicles would find difficulty in negotiating turns of 90° and over in the single lane widths at present recommended for use in the United Kingdom for radii of under 15m.

d) Cut-out appeared to be significant only when the load extended well forward of the front axle or when the semi-trailer on an articulated combination extended well forward of the king pin.

e) Relatively simple calculation of cut-in and cut-out (necessary to determine the required road width for a given turn) using generally available vehicle dimensions gave acceptable agreement with measured results.

In a report by Christie (15), the effect of vehicle size on lane width at a curve was reported. It showed a comparison between a short rigid vehicle and a long articulated vehicle while negotiating a curve. It was found that for the former its swept path is little wider than the vehicle, while for the latter its swept path gets wider as the vehicle progresses round the curve and the extent of the widening depends on the radius of the curve and its total angle.

Figure (1.2.1-1) compares the swept paths of a long articulated vehicle and a medium-sized family saloon car (4.2m long and 1.7m wide) when rounding two

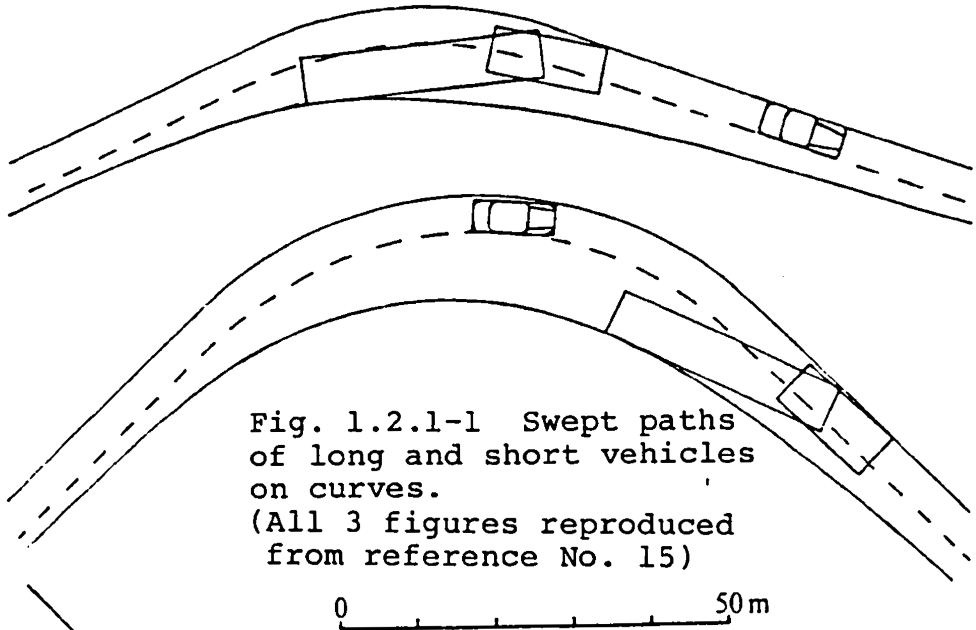

Fig. 1.2.1-1 Swept paths of long and short vehicles on curves.
(All 3 figures reproduced from reference No. 15)

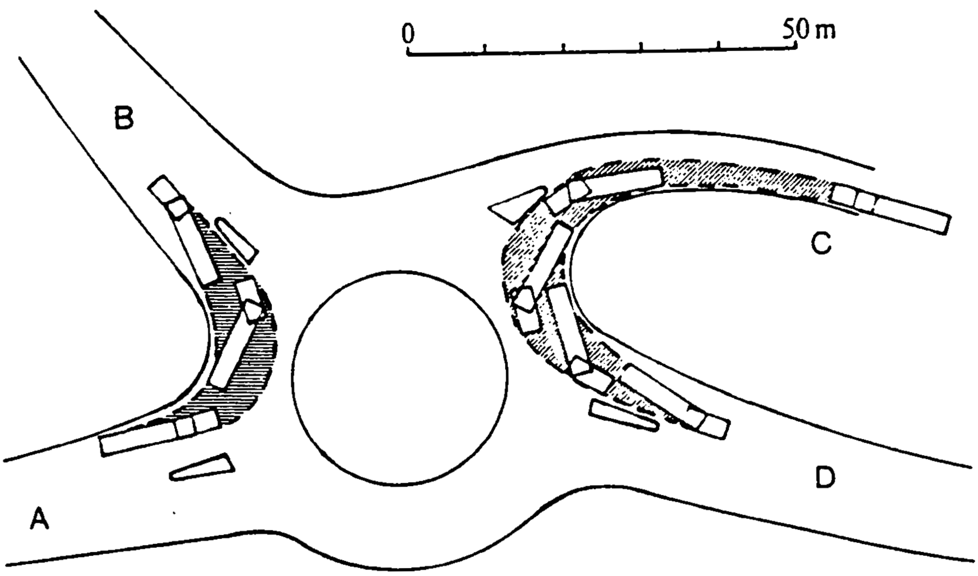

Fig. 1.2.1-2 Acute turns through roundabouts made by DTp articulated vehicle.

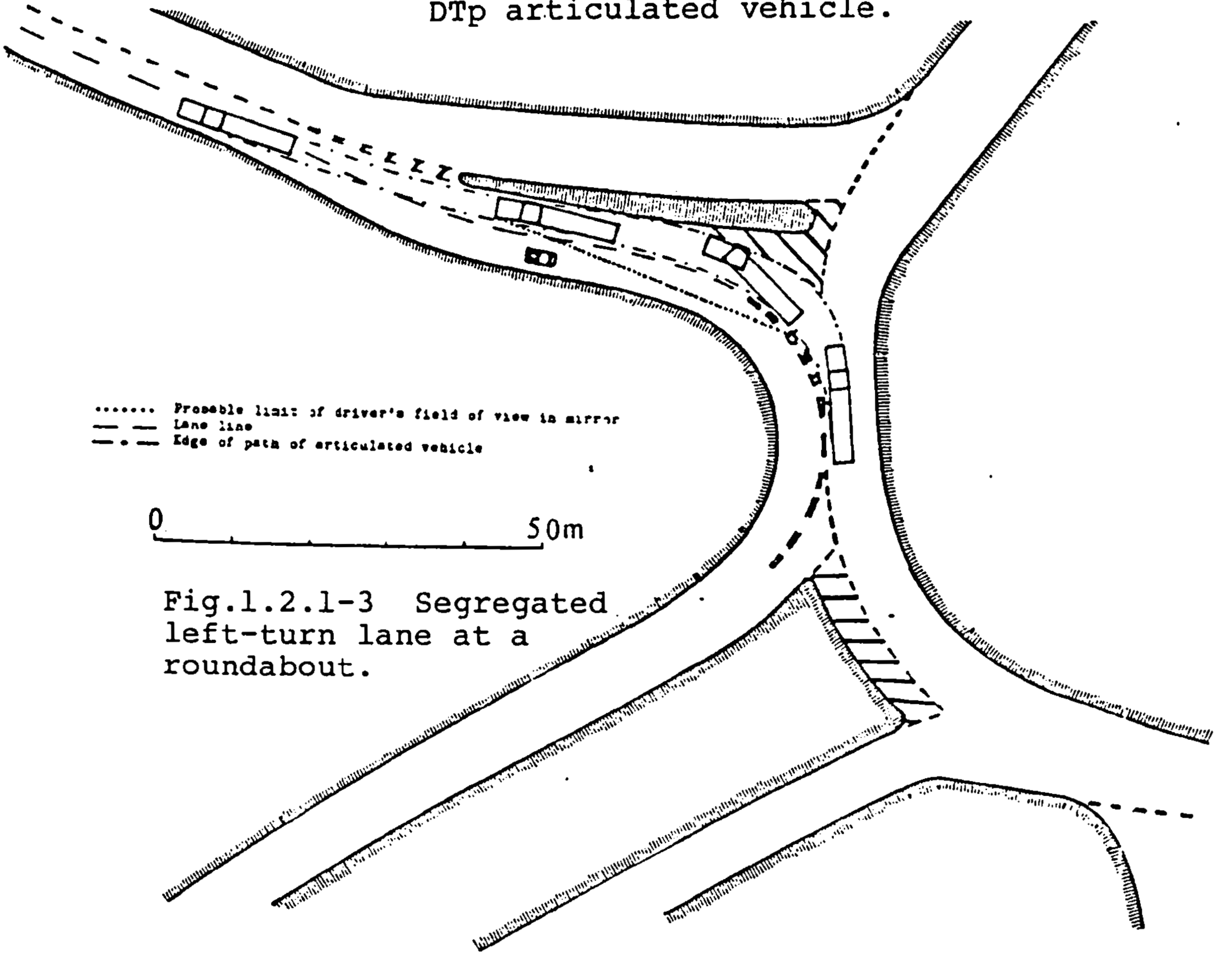

Fig.1.2.1-3 Segregated left-turn lane at a roundabout.

curves with an outside radius of 20m. The angles of the curves are 45° and 90°. The increase in the path width to 1.9m i.e. 0.2m wider than the saloon car, while the width of the path of the 2.5m wide articulated vehicle rises to 4.9m on the shorter curve and 5.8m on the longer curve. It is also suggested that on two-way carriageways the dividing line between traffic streams should not be in the centre of the carriageway (except at one point).

At roundabouts the problem of acute turns is illustrated by the report. Vehicle paths are shown at a particular roundabout (Figure 1.2.1-2) which is fairly large and has a central island diameter of 28m and an inscribed circle diameter of 50m. It was shown that the vehicle will require more width than the marked lane width and may only leave just enough space for a car to squeeze through. The report also shows that in order to avoid a collision when a long vehicle turns it is necessary to increase the entry width by providing a segregated lane as indicated in Figure (1.2.1-3).

1.2.2 - Turning circles at junctions

The fundamental requirement of the geometric design of junctions is to accommodate vehicle movements, a requirement which is influenced by vehicle dimensions. However, the composition of the traffic varies considerably, so it is difficult for a designer to determine suitable paths for vehicle manoeuvres,but when designing for vehicle circulation it is essential to allow for the most efficient movement of vehicles which permits the best use to be made of capital investment.

Design vehicle criteria can be used to solve this problem, where the design engineer will select for design the largest vehicle that is expected in significant numbers in the design year. This will require certain vehicle dimensions and operating characteristics which will determine the elements of geometric design. The low-speed turn at intersections is considered to determine the turning radii which are influenced by vehicle size, where the radius is controlled by the minimum turning paths of the vehicles. The minimum turning radius depends on the design vehicle which has a range between 24ft (7.32m) for passenger vehicle to 55ft (16.76m) for a semi-trailer combination of 50ft (15.24m) wheel base. When a vehicle turns at low speeds, the rear wheels track the front wheels on a shorter radius and the difference between the radii of the rear and front wheels is known as off-tracking. Off-tracking is dependent on the turning radius and the turning radius and the vehicle wheelbase. Combination vehicles involve two or more wheelbase lengths and off-tracking data are normally obtained by using scale models (17), (18).

Newland (19) has shown the geometry of the off-tracking of the rear wheels. It was suggested that the rear wheels do not follow in the tracks of the front but apparently swing towards the centre of the circle, a reason why large vehicles sometimes strike the kerb with their rear tyres when making tight radius turns.

Figure (1.2.2-1) shows this geometry and Figure (1.2.2-2) shows the turning path of the 24 to articulated design vehicle when steering to the same outside curve as the eight-wheeler in Figure (1.2.2-1). The work was a preliminary analysis to produce the minimum radius turning path for a 24 ton commercial vehicle at crawl speed. It was also indicated that further data was necessary for curve design to include all types of vehicles at different operating speeds.

Henderson and Cole (20) investigated the standard design car turning movement. In their investigation

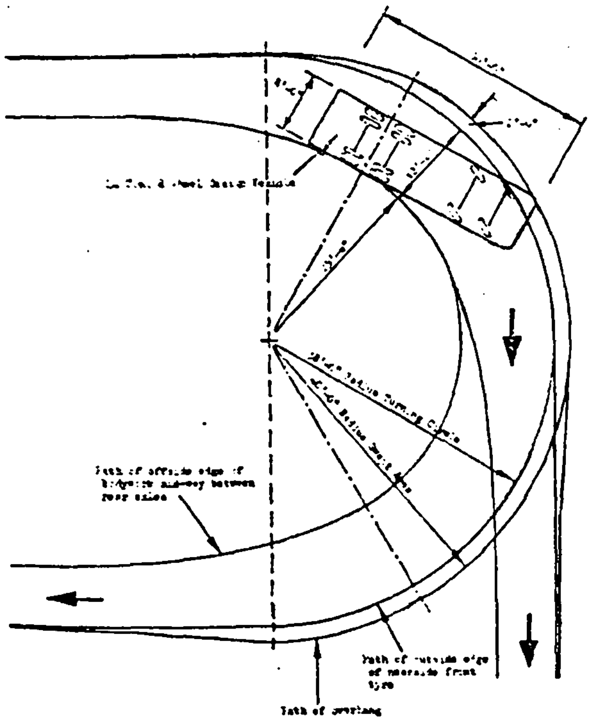

Figure 1.2.2-1 Minimum turning diagram for the 24 ton, 8 wheel, British design vehicle.
(Reproduced from reference No.19)

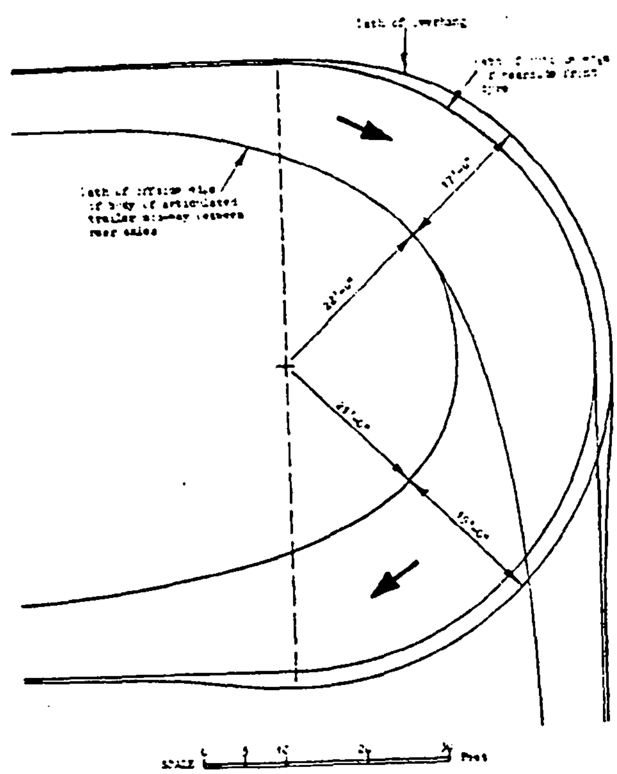

Figure 1.2.2-2 Turning diagram for 24 ton articulated British vehicle when turning to the same outside curve 8 wheeler shown in Fig. 1.2.2-1.
(Reproduced from reference No.19)

the geometrical shape of a number of vehicles of different length and wheel base was considered to negotiate a 90° corner and U-turns of different diameters. They showed how the required turning radius is affected by the size and type of the vehicle. Their conclusions were as follows:

a) The inner line of the track for a car turning both a 90° corner and a 180° U-turn, is not circular but of spiral form. Also the curves do not start to flatten out until the vehicle has passed the point where the front wheels start to straighten.

b) That both the length and width of the road required to straighten curve are less for the 26 ft (7.93m) vehicle than for the 36 ft (10.98m) vehicle.

c) When an articulated vehicle turns a corner it takes more space on the inner path than the maximum rigid vehicles.

d) The main points of turning through a 180° curve are similar in principle to those involved in the 90° turn.

Figures (1.2.2-3), (1.2.2-4) and (1.2.2-5)

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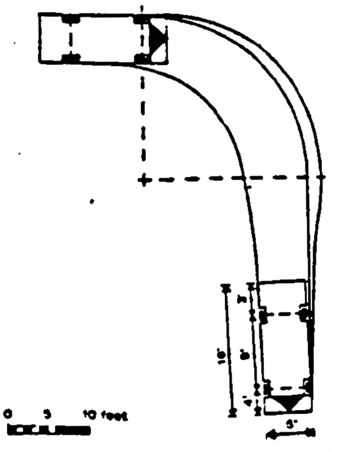
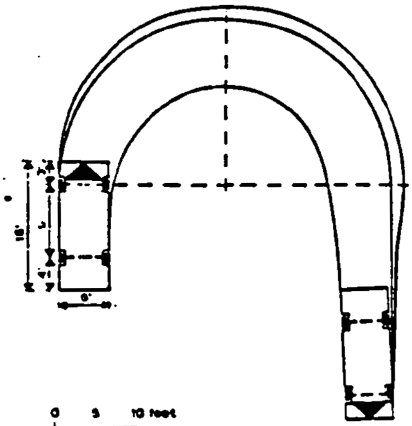
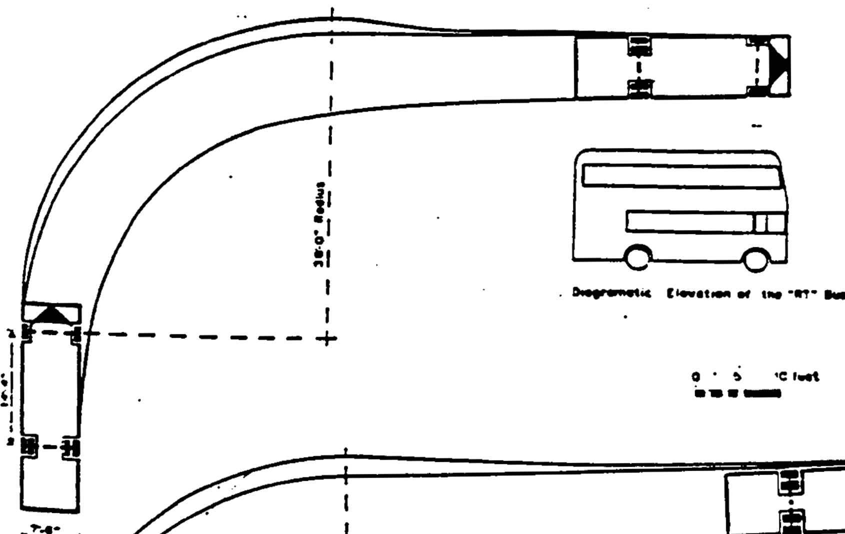


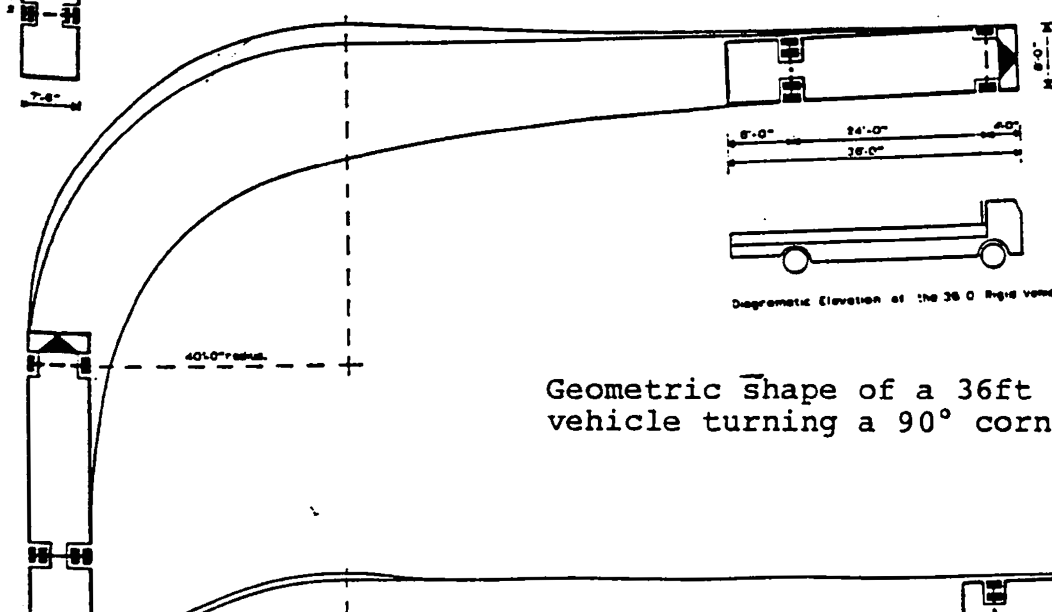
Geometrical shape of a 16ft x 6ft car turning a 90° corner.



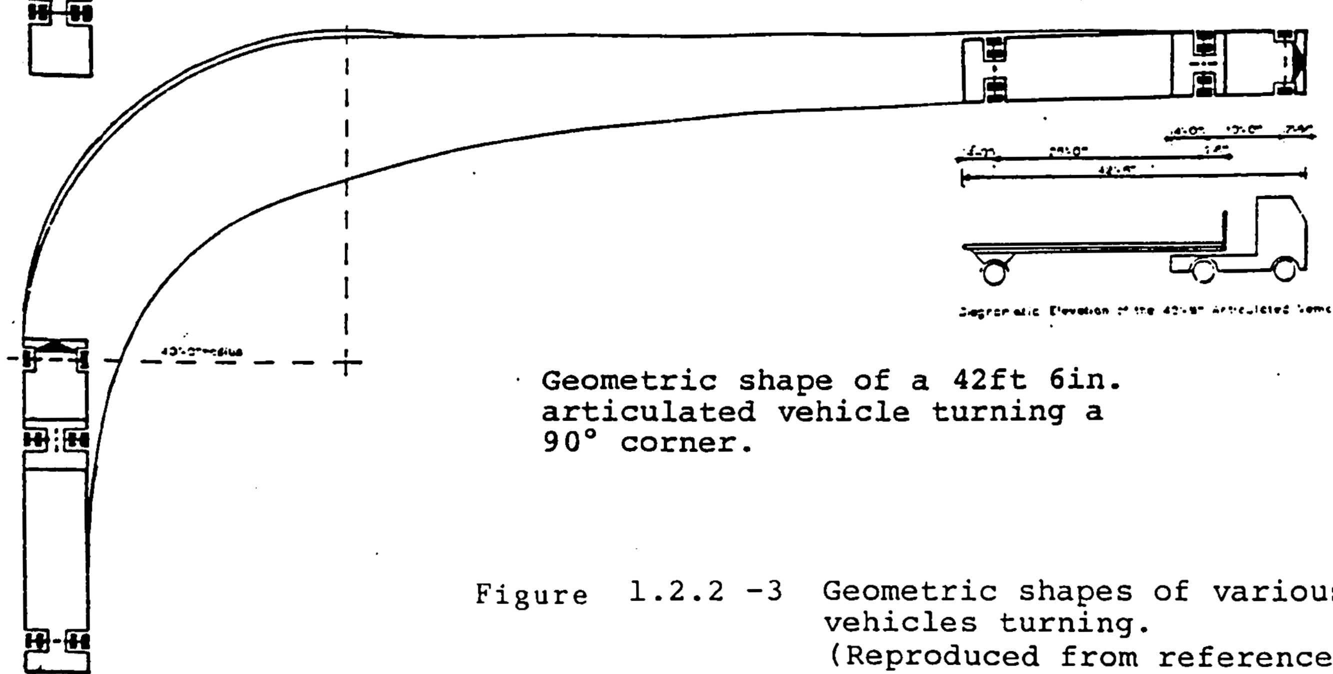
Geometrical shape of a 16ft x 6ft car turning a 180° U-turn.



Geometric shape of a 26ft RT bus turning a 90° corner.

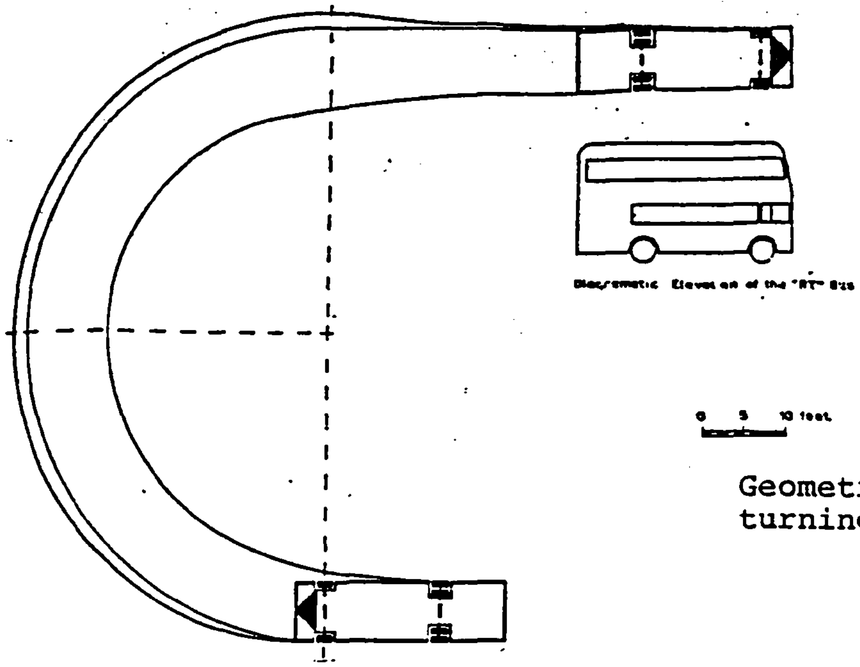


Geometric shape of a 36ft rigid vehicle turning a 90° corner.

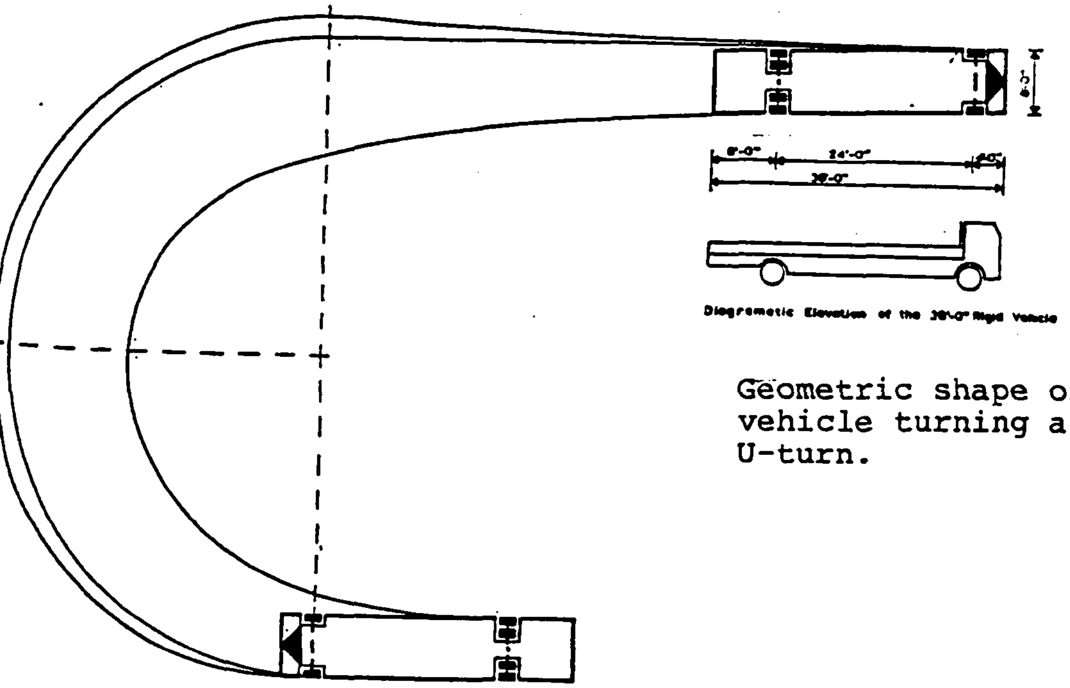


Geometric shape of a 42ft 6in. articulated vehicle turning a 90° corner.

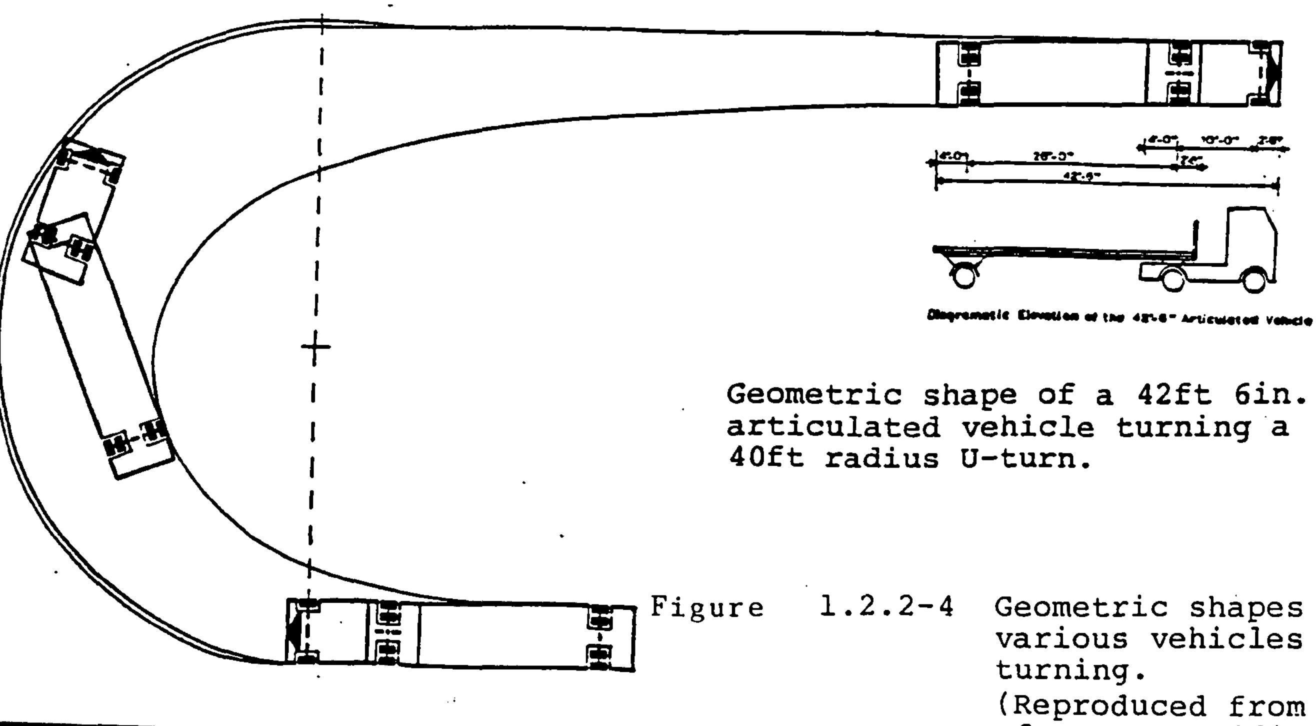
Figure 1.2.2 -3 Geometric shapes of various vehicles turning.
(Reproduced from reference No.20



Geometric shape of a 26ft RT bus turning a 76ft diameter U-turn.



Geometric shape of a 36ft rigid vehicle turning a 40ft radius U-turn.



Geometric shape of a 42ft 6in. articulated vehicle turning a 40ft radius U-turn.

Figure 1.2.2-4 Geometric shapes of various vehicles turning.
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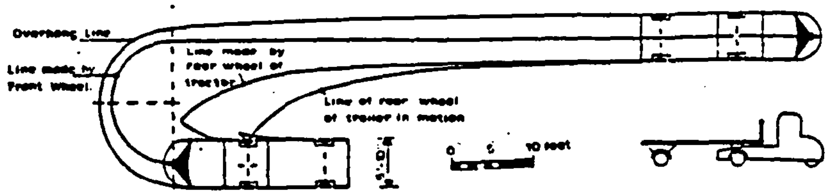
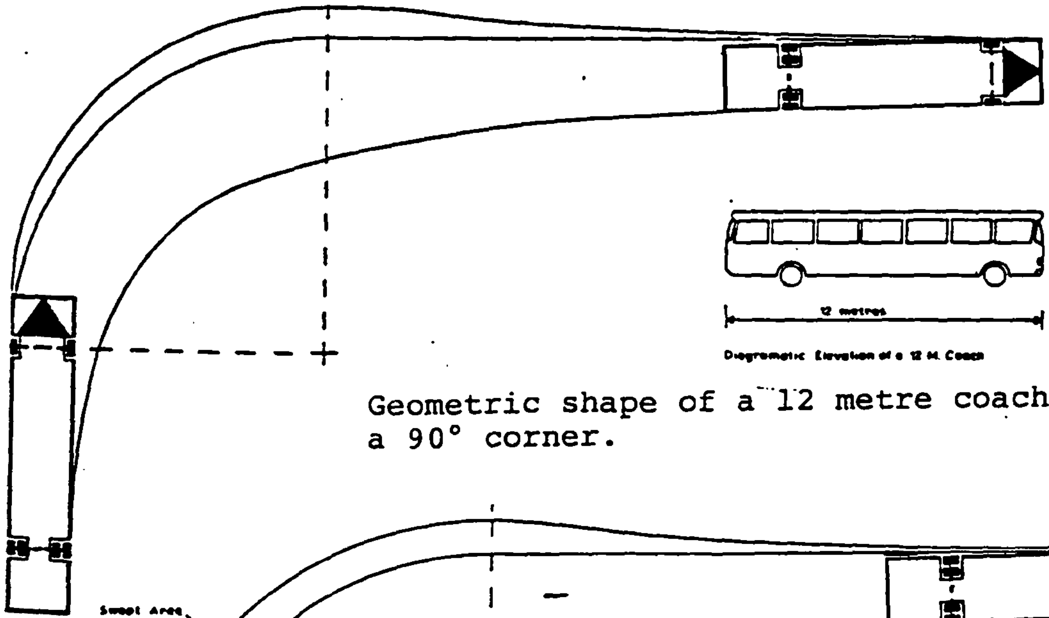
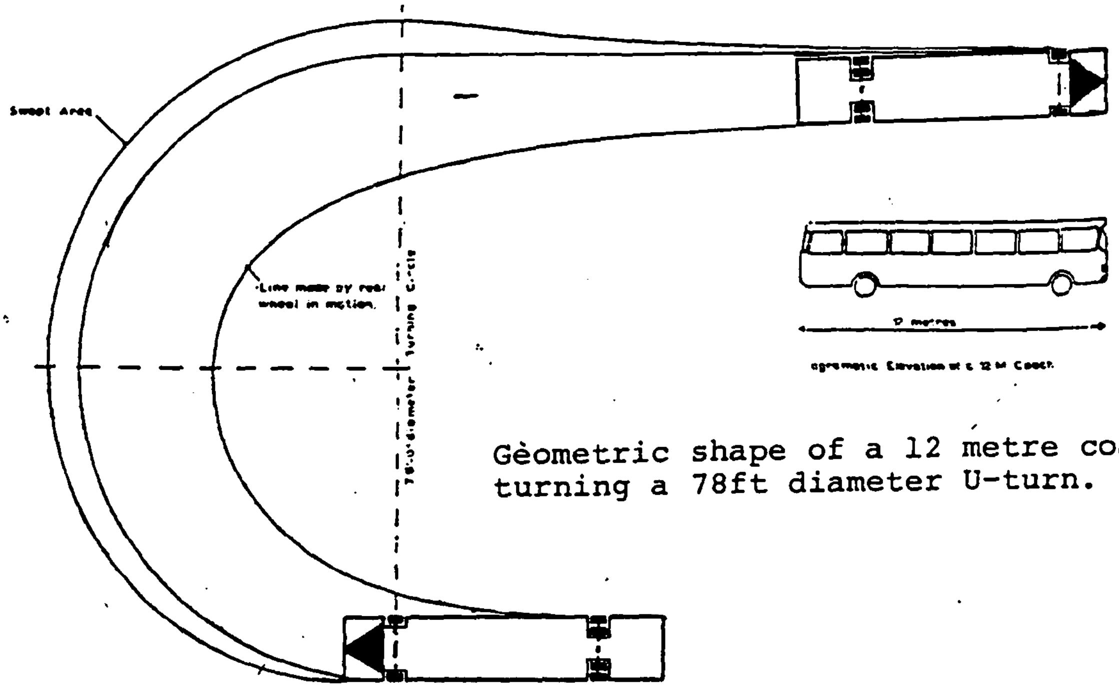


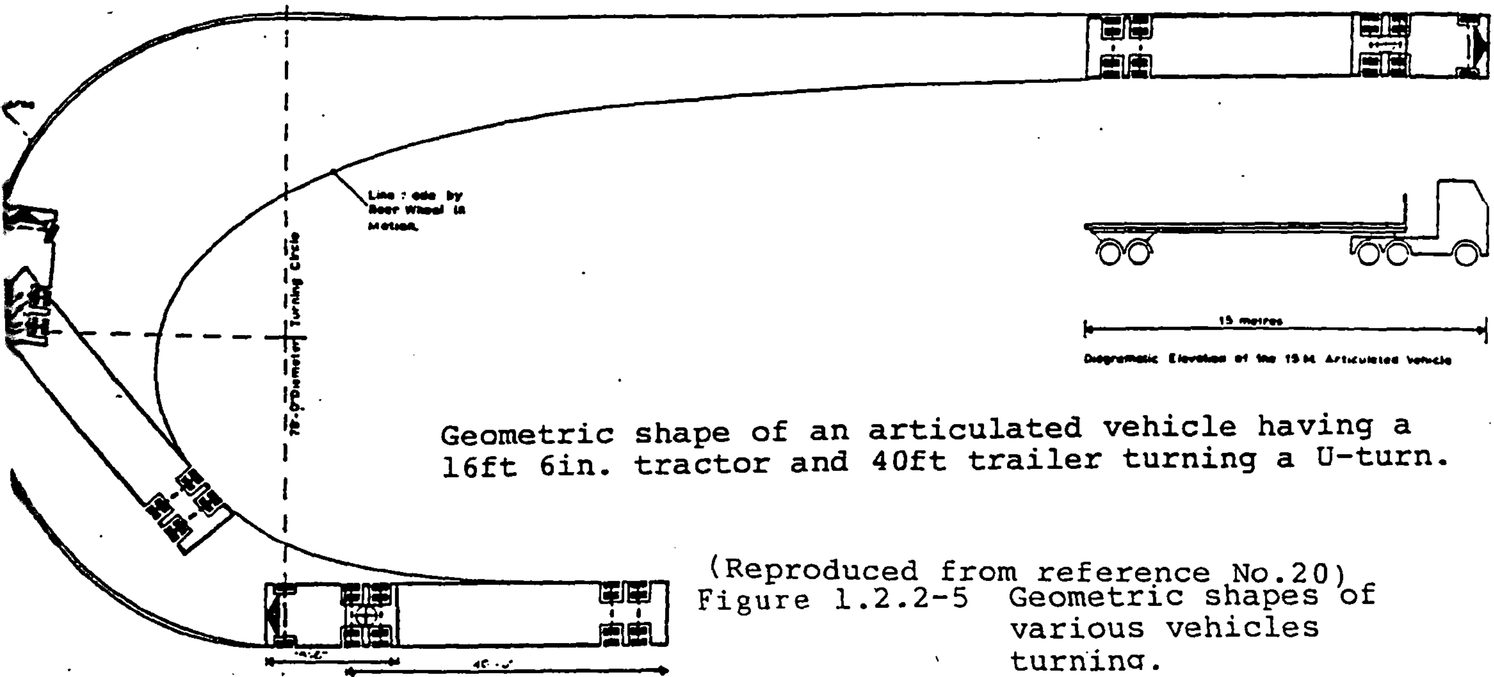
Geometric shape of a Scammel Townsman turning a 20ft U-turn.



Geometric shape of a 12 metre coach turning a 90° corner.



Gèometric shape of a 12 metre coach turning a 78ft diameter U-turn.



Geometric shape of an articulated vehicle having a 16ft 6in. tractor and 40ft trailer turning a U-turn.

(Reproduced from reference No.20)
Figure 1.2.2-5 Geometric shapes of various vehicles turning.

show these points and give some comparison between different types of vehicles in their manoeuvres criteria. The American Association of State Highways Officials (21) used for the geometric design of rural highways seven design vehicles, one of which is selected with dimensions and turning characteristics equal to or greater than those of the largest vehicles expected in appreciable numbers.

For urban highways and arterial streets,Table (1.2.2-1) shows the relationship between vehicle wheelbase, overall lengths, minimum turning radii and minimum inside radii. And the minimum edge of pavement designs for turns at intersections are shown in Table (1.2.2-2), in which five types of vehicles were introduced.

Design Vehicle		Dimensions (m) and Design Elements			
Type	Symbol	Wheel-base (m)	Overall length (m)	Minimum turning radius (m)	Minimum inside radius (m)
Passenger car	P	3.36	5.80	7.3	4.7
Single-unit truck	SU	6.10	9.15	12.8	8.5
Single-unit bus	Bus	7.63	12.20	11.6	6.4
Articulated bus	A-Bus	12.81	18.30	12.8	7.1
Semi trailer-Comination intermediate	WB-40	12.20	15.25	12.2	6.1
Combination-large semi-trailer - full trailer	WB-50	15.25	16.78	13.7	6.0
Semi trailer-full trailer combination	WB-60	18.30	19.83	14.7	6.9

Table 1.2.2-1 Relationship between vehicle dimensions and turning radii.
(Reproduced from reference No.21)

Angle of Turn (deg)	Radius (m)	Design Vehicle				
		P	SU	WB-40	WB-50	WB-55
30	R1	18.3	30.5	45.75	61.0	83.88
	R2	-	-	-	-	-
45	R1	15.25	22.88	36.6	-	-
	R2	-	-	-	36.6	42.7
60	R1	12.2	18.3	27.45	-	-
	R2	-	-	-	28.98	33.55
75	R1	9.5	16.78	-	-	-
	R2	7.63	13.73	18.3	19.83	27.45
90	R1	9.15	15.25	-	-	-
	R2	6.1	12.2	13.73	18.3	22.88
105	R1	-	-	-	-	-
	R2	6.1	10.68	12.2	16.78	19.83
120	R1	-	-	-	-	-
	R2	6.1	9.15	10.68	13.73	15.25
135	R1	-	-	-	-	-
	R2	6.1	9.15	9.15	12.2	13.73
150	R1	-	-	-	-	-
	R2	5.49	9.15	9.15	10.68	12.2
180	R1	-	-	-	-	-
	R2	4.58	9.15	6.1	7.63	7.63

where

R1 = Simple Curve Radius

R2 = Simple Curve Radius with Taper

Table 1.2.2-2 Minimum edge of pavement designs for turns at intersections.
(Reproduced from reference No.21)

Contd./....

Table 1.2.2-3 continued.

P	=	Passenger car
SU	=	Single-Unit Truck
WB-40	=	Semi trailer-Combination intermediate
WB-50	=	Semi trailer-Combination large
WB-55	=	Semi trailer-full trailer Combination

1.3 - Effect of vehicle type on capacity

1.3.1 - Between intersection situations

The composition of traffic flow is one of the main factors that affect capacity, as vehicles of different types require different amounts of road space because of variations in size and performance. The problem of variation can be overcome by using an adjustment factor, where the volume of mixed traffic is converted to equivalent passenger cars. Under all conditions the commercial vehicles take up more space than passenger cars and their presence within the traffic stream has to be considered in order to determine the number of passenger cars that each commercial vehicle is equivalent to under specific conditions. In addition to their space criteria, commercial vehicles have poorer operating capabilities than passenger cars, particularly with respect to acceleration, deceleration and the ability to maintain speed on upgrades. The operating capabilities are the most critical because heavy vehicles cannot maintain the same performance as passenger cars in many situations; large gaps form in the traffic stream that are difficult to fill by passing manoeuvres. This creates inefficiencies in the use of roadway space that cannot be completely overcome, which is apparent on sustained steep upgrades where vehicles' operating capabilities differ and on two-lane highways where passing must be accomplished by using the opposing travel lane.

Heavy vehicles have to operate in a low gear in down-grade operations, particularly where down-grades are so steep as to require that. In such cases, heavy vehicles again must operate at speeds slower than those of passenger cars and wider gaps will form in the traffic stream unless overtaking opportunities prevail.

Two major problems which effect capacity arise on links between junctions. The first problem is the situation when a long articulated vehicle passing a stationary vehicle disrupts other traffic more than a short rigid vehicle does in the same situation. One reason for this is that after the tractor has pulled out to the right and has begun to move parallel to the axis of the road, the trailer is only gradually pulled into line with it. The outward movement must therefore be started well in advance of the stationary vehicle. Similarly, the passing vehicle cannot move back to the left until the tractor is well past the stationary vehicle, otherwise the rear of the trailer will cut in too soon and strike some other vehicle.

This problem was discussed by Christie (15) in which he showed a comparison between a passing distance of a 15.5m long articulated vehicle and a corresponding manoeuvre by a short rigid vehicle (car or light van), 4.1m long. It was noted that for the former the passing distance as measured by the total length of centre-line cut by any part of the passing vehicle was about 56m while for the latter was 28m. This variation is shown in Figure (1.3.1-1).

The second problem is when overtaking moving vehicles where the passing distance is increased further.

The distance required is;

$$D = SV(V-V') - L \text{ metres (approximately)}$$

where

V and V' are the speeds in Km/h of the overtaking and overtaken vehicles respectively.

S is the distance (in metres) travelled by the overtaking vehicle relative to the overtaken vehicle.

L is the length of vehicle.

If, for example, V = 100 Km/h and V' = 90 Km/h then D = 316m for the short vehicle and D = 674m for the long vehicle using S = 32m for short vehicle and 69m for long vehicle.

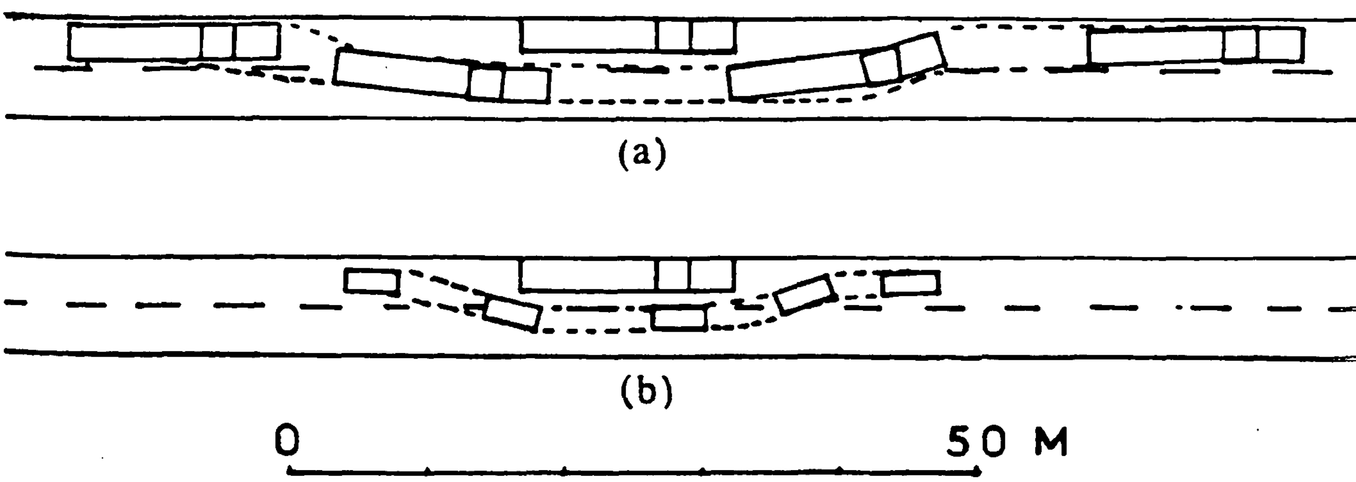


Figure 1.3.1-1 Passing a stationary vehicle, 15.5m long, on a two-way, two-lane road of width 7.3m. Passing vehicles:

(a) DTp articulated vehicle, 15.5m long

(b) Short rigid vehicle (car or light van), 4.1m long

(Reproduced from reference No.15)

1.3.2 - At priority junctions

The priority situation represents the most common type of intersection control in Great Britain and indeed in many other countries. Figures (1.3.2-1) and (1.3.2-2) show the 3-arm or "T-junction", the 4-arm lay-outs (cross intersection) and the 4-arm staggered inter-sections are the most frequent type of intersection under which the priority situation operates to solve the con-flict between merging and crossing vehicles. The use of "Give Way" and "Stop" control at intersections has considerably increased the number of occasions at which a driver has to merge or cross a major road traffic stream making use of gaps or lags in one or more conflicting streams.

In a report by the Transport and Road Research Laboratory (22),it was stated that in relation to traffic effects, it is important to recognise the essential asym-metry in the traffic flows of major/minor junctions. The level of major road flows consistent with satisfactory operation of the junction (i.e. still allowing an accept-able inflow from the minor arm) do not usually approach the capacity limitations of the major road itself. Thus composition effects in both major and minor flows are only relevant in as much as they affect the minor road capacity. The minor road flows are affected by the composition of the major road streams or by the minor road traffic itself.

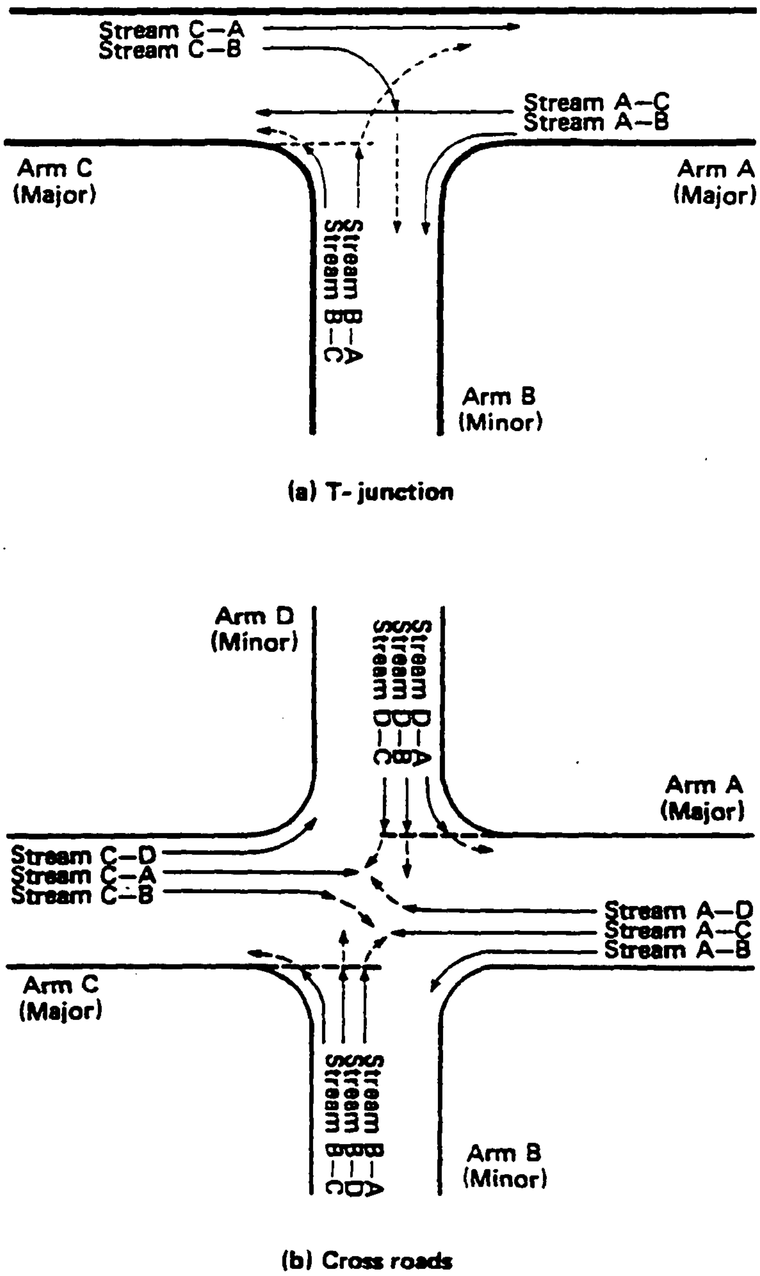
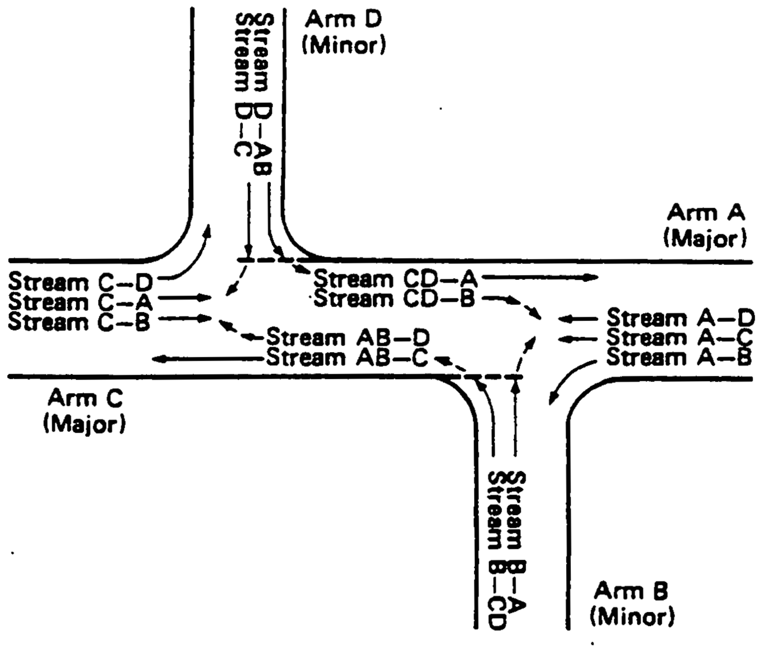
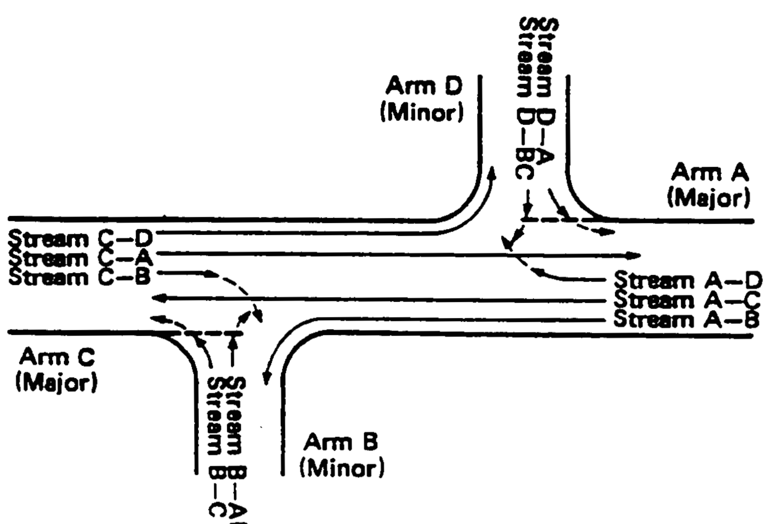


Figure 1.3.2-1 Traffic streams at 3 - and 4 - arm major/minor junctions.
(Reproduced from reference No.23)



(c) Left-right stagger junction



(d) Right-left stagger junction

Figure 1.3.2-2 Traffic streams at stagger junctions.
(Reproduced from reference No.23)

It has been established that the right-turning minor road capacity depends on the total vehicular flows in the relevant major road streams. Any effect due to composition should appear as an associated factor between the fraction of heavy vehicles in a major road stream and the flow counts for the minor road movement (22).

The association between the minute-by-minute composition of the major road stream and the minor road flow was tested by the Transport and Road Research Laboratory (22). The tests showed that although there was an appreciable proportion (10-15%) of heavy vehicles at all but one site, none of the tests revealed any significant association. So they have stated that there was no evidence to suggest that the composition of the major road streams has any detectable effect on the emerging minor road streams.

The effects of traffic composition on the capacity of junctions are often represented by PCU factors. It was suggested (22) that PCU factors would differ from unity only for the minor road heavy vehicles and would take values which are strongly site-dependent.

1.3.3 - <u>At roundabouts</u>

In the design of roundabouts there are many requirements to be satisfied such as the physical features and vehicles manoeuvres. The wide range of size of vehicle to be catered for brings further problems. A long vehicle may have to take a different course, both on approach to and in the roundabout.

The Department of Transport's manual, Driving (39), under "dealing with roundabouts" says:

> "The course to be followed by long vehicles and buses will depend on the shape and size of roundabouts and the amount of room available. Long vehicles, especially articulated ones, need much more room than cars and may be forced to take up positions which appear wrong to you. Don't be tempted to squeeze through gaps which could suddenly disappear. Under "Signals at roundabouts" it says:
>
> "Watch out, too, for long vehicles - they may have to take a different course both on the approach to the roundabout and when going round it to allow for the rear of the vehicle cutting in".

In small roundabouts with central islands of 4m diameter and minimum outer dimensions of about 28m the DTp articulated vehicle can make all the turns through a roundabout with these dimensions and still leave a clear space of 1m or more adjacent to the central island and the perimeter (15).

The manoeuvring of long vehicles at large roundabouts may cause some problems due to a combination of the following three factors: the greater length of long articulated vehicles; their low acceleration when fully laden; and the high circulatory speeds, typical of very large roundabouts. A driver of a long articulated vehicle, when entering a roundabout, has to wait at the give-way line until there is no vehicle visible to his right before starting to move forward. The time required for this driver to move forward in most cases is longer than the time required for other vehicles in the circulating carriageway to arrive. Other drivers experience the same problem, but to a lesser degree.

The capacity of offside roundabout entries has been studied by the Transport and Road Research Laboratory at twenty-eight public road sites (24). The effects of traffic composition and of geometric factors have been evaluated.

Passenger car units were derived for heavy vehicles and two-wheeled vehicles: rounded to the nearest integer, the average values were:

heavy vehicles	entering	two
	circulating	two
two-wheeled vehicles	entering	zero
	circulating	one

1.3.4 - At signalized intersections

The capacity of signalized intersections expressed in vehicles per hour of green time, is the reciprocal of the average headway between vehicles during saturation flow. This suggestion was discussed by Webster and Cobb (25) in 1966 and their method was quoted by the Ministry of Transport in 1965 and 1966, who gave a nomogram to assist in calculating capacity.

Stewart (26) estimated the average headway when the traffic stream is composed entirely of standard cars to be as 2.21 seconds and the resulting capacity was 1,630 vehicle/hour of green time. An average headway of 1.925 seconds for small cars following small cars gave a capacity of 1,870 vehicles per hour of green for a homogeneous stream of small cars. This represented a 15% increase in capacity when a traffic stream of full-sized cars was replaced by small cars.

It was also suggested that measurements of headway between vehicles being discharged from a queue at signalized urban intersections showed vehicle size to have an effect on headways during saturation flow. The shortest headways were found when the preceding and following vehicles were both small cars. The difference in headways between small cars and full-sized cars became insignificant as the traffic stream approached free-flow conditions. It was also suggested that the effect of small cars on intersection capacity was greater when a large number of turning movements were made.

Branston and Gipps (27) analysed the elapsed time from the instant the lead car started to move to the time the last car of the platoon crossed the starting position of the lead car. In their study the crossing time for the first 10 vehicles discharged from a saturated signalized intersection during the green phase were observed in street traffic to have a mean of 16.98 seconds at a point near the stop line. The mean crossing time per vehicle for cruising speeds of 20-50 mile/hour was 1.88 seconds and 1.69 seconds for standard and small cars respectively. This suggested a saturation flow of 1,860 and 2,056 vehicle/(hour of green cycle time) for standard and small car platoons respectively. These saturation flows were calculated using a time of 2.72 seconds for the response time of the lead car to the onset of the green phase. The duration of the green phase was assumed to be 30 seconds. The 10.6 per cent difference in these estimated saturation flows was consistent with previous estimates. It was suggested that the difference in the travel times of the last car to the crossing point could be more directly related to the vehicle length and performance. The average inter-vehicle separation adopted by the drivers when the platoon was stationary was found to be approximately the same for both the standard car and small car platoons. As a result of the vehicle length differences, the last car in the small car platoon was 174 ft (52.78m) from the starting position of the lead car, compared to 213 ft (64.61m) for the last car in the standard car platoon. The added travel distance of the last standard car was, in turn, partly compensated by a higher acceleration.

Branston (28) analysed the variation of PCU values using three vehicle types: buses, medium commercial vehicles (lorries with two axles) and heavy commercial vehicles (lorries with three or more axles). It was indicated in his investigation that the dependence of PCU values of commercial vehicles on saturation flow was of interest from a behavioural viewpoint, but a simpler approach which uses average PCU values was more appropriate for practical applications.

Vehicle type effect was apparent when straight-on cars were constrained to follow behind a left-turning car through the junction, the latter's PCU value was, on average, 1.33. PCU values of left-turning buses, medium and heavy commercials were about 1.2 times the corresponding values for straight-on vehicles of these types.

It was also suggested by Branston that unless the proportion of motorcycles or pedal cycles in the traffic stream was greater than about 20 per cent, or such vehicles interfered severely with left-turning traffic, they would have very little effect on saturation flow and could be ignored for practical purposes.

1.3.5 - At Grade Separated Intersections

The criteria on which the justification of the provision of grade separation is based is mainly the cost of vehicle delay. However, the provision of a smooth uninterrupted flow and reduction in the dangers on high speed routes may also be considered in that justification.

In a report by the Transport and Road Research Laboratory (29), the entry capacities of seven entries to grade-separated roundabouts were investigated. All entries were fed by exits from motorways and their capacities were compared with those predicted by a 'unified' formula for at-grade roundabouts.

In their study it was found that traffic performance differed slightly from that at-grade roundabouts and the entry capacity was more sensitive to the circulating flow. A slightly modified form of the unified capacity formula (29) for at-grade roundabouts was thus suggested for use with grade-separated roundabouts. The modified form is:

$$Qe = 1.11F - 1.40\ fc\ Qc \quad \text{PCU/h}$$

where F and fc are as calculated from unified formula (29).

The formula is applicable at all grade-separated roundabouts where:

i) motorways connect directly with, or terminate at, the roundabout and;

ii) exit slip-roads from motorways or dual carriageways pass above or below the centre of the roundabout.

In the study vehicles were classified as 'light' (those with 3 or 4 tyres) and 'heavies' (those with more than 4 tyres - including cars pulling trailers or caravans). The number of two-wheelers was negligible at all of the sites.

All pairs of entry and circulating flows at each site for saturated minutes were converted to passenger-car units per hour (PCU/h) using the factors:

1 light per minute = 60 PCU/h

1 heavy per minute = 120 PCU/h

Recent studies in the USA have been directed towards the calibration of passenger car equivalents.

Roess and Messer (30) investigated the passenger car equivalents for uninterrupted flow, they analysed capacity based on performance parameters so PCE values should relate to those same performance parameters. It was found that speed was the principal criterion for designation of levels of service. Thus, conversions from mixed to PCE volumes would not alter the performance parameters defining level of service. It was suggested that if this principle was to be extended to the uninterrupted flow procedure of Circular 212 (31), PCE's should be based on equal densities, because density is the principal parameter defining level of service. It was noted that none of the other concepts for PCE's reviewed in their research guarantees that the equivalent PCE volume operates at the same performance levels as the actual mixed traffic stream.

The constant volume-to-capacity (V/C) ratio approach used in developing the Circular 212 values was suggested to be relevant in its own right, because V/C values are related to speed and densities. Further, the proportion of capacity used and the proportion still available are critical pieces of information. However, while V/C ratios were held constant, it was suggested that equivalent traffic streams might not operate at the same speed and density as the actual mixed traffic stream.

The use of weight-to-horsepower ratio was introduced as a criteria in vehicle classification. The following recommendations were made:-

i) The passenger car equivalent values for normal truck populations should be based on performance characteristics of a 200 lb/hp truck.

ii) For non-standard truck populations the passenger car equivalent values should also be provided (light truck populations would be represented by a 100 lb/hp truck and heavy truck populations by 300 lb/hp truck).

1.4 - Effect of vehicle type on delay

1.4.1 - Introduction

Intersections represent the main locations of the highway system where delay to vehicles movement is to be considered. The magnitude of this delay is influenced by intersection design and vehicles performance, where the latter is related to vehicle types. A passenger car is likely to negotiate the intersection geometric layout with less effort than the heavy goods vehicles and buses.

The areas in which vehicles type could affect delay at highway intersection are:

1 - The amount of starting delay inherent in the starting movement of a queue of stopped vehicles.

2 - Delay due to the density of traffic flow.

3 - Delay due to vehicle waiting for a suitable gap in the opposing stream while turning.

All these areas could be reduced if the percentages of heavy commercial vehicles were reduced, as they have low ability to manoeuvre at intersections.

Junctions are complex areas of traffic intersection, their physical characteristics (number of lanes, gradients, geometric layout, location of bus stops and pedestrian crossings), traffic use (flows and turning movements, classifications, performance speeds, route types and pedestrian flows) and the form of traffic control (signals, channelisation, roundabouts, turning restrictions and grade separation) all influence the nature and amount of delay distributed among users.

1.4.2 - At signalized intersection

Delay to vehicles at signalized intersections has been analysed by a large number of researchers. Their study approaches included mathematical analysis and traffic simulation.

Allsop (32) has made a comprehensive survey of expressions for the average delay at fixed time traffic signals and Hutchinson (33) has compared numerical expressions derived by Webster (25), Miller (34) and Newell (35). His numerical comparisons were made by taking as a base Webster's full expression which was derived by computer simulation.

Other researchers have analysed the effect of composition of traffic upon delays. Sosin (36) investigated the changes in the composition of surveyed traffic and used delay to introduce comparable equivalents for various kinds of vehicles. The equivalents referred to the influence of the composition of traffic upon delays. From the calculations the following equivalents were obtained:

Passenger cars	1.0
Lorries	1.6
Buses	1.7

Lorries with trailer and articulated vehicles	2.8
Motor cycles and bicycles	0.6

Kimber, McDonald and Hounsell (37) indicated that the role of PCUs in the saturation flow calculation is an estimate of the number of passenger cars that would pass if there were no heavy vehicles, or of the number of vehicles that could pass at a given traffic composition. They have assumed that PCU values should be chosen so as to optimise the saturation flow prediction in some way. However, although it is an important parameter of design, saturation flow is not the ultimate measure of junction performance.

In order to minimise vehicle delay the signal timings have to be optimised. The saturation flow and the degree of saturation are only intermediate variables in this process and PCU values could be chosen so as to optimise the signal-setting process (to minimise delay) rather than to maximise the accuracy of saturation-flow prediction. Therefore, some simple simulations of queueing at a traffic signal have been carried out in order to investigate the relationship between those PCU estimates which were chosen to produce signal settings for minimum delay, and those which were obtained by the headway ratio method and by asynchronous regression.

The ratio of the mean release headway of cars and heavy vehicles is a fundamental criterion of the traffic. The signals settings - cycle time, effective red time and ratio of green times - are the design parameters to be adjusted so as to achieve optimum conditions. In order to arrive at these settings a PCU value has to be assumed for heavy vehicles. This value can be chosen so as to yield settings which minimise the total vehicular delay for a given percentage of heavy vehicles (37).

1.4.3 - At priority intersections

The growth of traffic as a whole and the increase in the number of heavy commercial vehicles have led to considerable congestion and delay at priority intersections during peak hours. This has resulted in making the prediction of queue lengths and vehicular delay important in many aspects of traffic engineering.

Considerable work has been carried out using empirical, mathematical and simulation processes to investigate delay at priority intersections. At the present time the average delay to minor road vehicles is calculated using expressions introduced by the Transport and Road Research Laboratory (38).

McDonald, Hounsell and Kimber 1984 (37) investigated geometric delay at priority intersections by grouping vehicles into two types, light vehicles and other goods vehicles (heavy goods vehicles). They have indicated that heavy vehicles were found to be subject to delay in excess of that predicted for light vehicles. It was 2.1 seconds at priority junctions (approximately 25 per cent of the delay averaged over all delayed movements). The geometric delays are classified in table 1.4.3-1 for light vehicles.

Delay (sec.) - light vehicles *					
Left turn		Right turn		Straight ahead	
Side Road	Main Road	Side Road	Main Road	Side Road	Main Road
7.8	5.7	10.6	6.5	12.2	0

* Add 2.0 seconds if mean link speeds > 65 K.p.h.
Add 1.4 seconds if visibility requirements do not meet the requirements of reference (37).

Table 1.4.3-1 Geometric delays for light vehicles at priority junctions, 1984.

1.4.4 - At roundabouts

At any approach in a roundabout the build-up of vehicle queue is governed by the rate of discharge and the rate of arrival. The rate of discharge depends upon the time interval between vehicles already in the circulating section which is related to traffic flow and its composition in that section. Vehicles' delay as a result of this build-up is referred to as queueing delay. It is calculated by noting the queue length at the beginning of a time segment and the queue length at the end of the segment.

The other form of delay is geometric delay caused to a vehicle due to the presence of the roundabout. It occurs because vehicles have to reduce their speed to negotiate the junction, deviate from the direct path which would be available if the roundabout was not there and accelerate back to normal running speed.

McDonald, Hounsell and Kimber, 1984 (37) introduced delay formulae based on extensive observations on public roads. The delay for each vehicle making a particular movement is calculated as the difference between:

(i) the time taken to travel through the junction between the points where deceleration begins and acceleration ends and

(ii) the time taken to travel between these two points in the absence of the junction.

Both these depend on approach and departure speeds and on certain geometric parameters of the roundabout. Heavy vehicles are attributed 15 per cent greater geometric delay than light vehicles. This results from their lower acceleration/deceleration capabilities and slower speeds round curves.

In their studies of geometric delay, two classifications of vehicle were used, light vehicles and heavy vehicles. The variation in vehicle type was accompanied by a correspondingly wide range of geometric delay measurements for the classification of heavy vehicles.

For grade-separated roundabouts table 1.4.4-1 shows delay (sec.) for light vehicles.

Delay (sec.) - light vehicles *		
Left turn	Right turn	Straight ahead
10*	28*	11

* Add 3 seconds for flyover when travelling from minor road to motorway.

Table 1.4.4-1 Delay for light vehicles at grade-separated roundabouts, 1984.

Their regression equation for all roundabout was

$$g = 0.07ED+1.12(Y-V_{BC})+3.62$$

where

g = delay (sec)

ED = extra distance involved in negotiating the intersection (m)

Y = average of approach and exit link speeds (m/sec) = $(V_A+V_D)/2$

V_{BC} = average speed within intersection

The geometric delay for heavy vehicles was estimated as per site categories:

Intersection type	Delay to heavy vehicle
Free-flowing motorway links	As light vehicle
Priority junction, diamond intersection	As light vehicle + 25%
All other intersections	As light vehicle + 15%

The geometric delay will be supplemented for capacity limited streams by queueing delays as traffic flows increase. It is as well to note that it includes elements of delay associated with the driver checking that it is safe to enter the junction.

The work covered a wide area of geometric delay and considered two different approaches to the estimation of this delay, one used the standard techniques of multiple regression or category analysis and the other approach made use of a synthetic model. It represents an important result because it gave the latest information on delay variation due to type of vehicles present in the traffic system.

2

Effects of Vehicle Type at Traffic Signals

CHAPTER TWO

Effects of vehicle type at traffic signals

2.1 - Introduction.

The signalized intersection is one of the most complex locations in a traffic system. An analysis of its operational characteristics must consider the effect of its geometric design, signal operation and traffic factors. The latter includes the pattern and composition of arriving traffic, turning movements, presence of pedestrians and general driver characteristics. The pattern of traffic arrivals is strongly influenced by nearby traffic signals and their co-ordination.

Capacity of an approach is affected by vehicle types especially buses and articulated vehicles with their lower acceleration ability and other operating characteristics such as deceleration and turning movement requirements. This variation in vehicle types led to the application of a conversion procedure in order to obtain a standardised flow rate at each approach of the signal intersection. In the British method (25),the following factors were used to obtain the passenger car unit equivalents at a traffic signal approach:-

1	Bus	= 2.25	PCU
1	Heavy and medium commercial vehicle	= 1.75	PCU
1	Light commercial vehicle	= 1	PCU
1	Motor cycle, moped or scooter	= 0.33	PCU
1	Pedal cycle	= 0.20	PCU

The United States method applies an adjustment factor that reduces or increases the capacity and service volumes which will convert the base condition of 5 per cent trucks and through buses to any existing percentage (40).

Adjustment factor = 1 -1 0.01 (T - 5)

where

T is the percentage of trucks and through buses.

This value is based on observations of performance with truck percentages below 20%. Gwynm (41),suggested that this overestimates the effect of trucks when they constitute over 20% of approach traffic.

Most of the delays occur at road junctions which may substantially reduce traffic flows when controlled by traffic signals. In central London even during off peak hours over one third of a typical journey is spent at traffic signals (42).

In this chapter the effect of vehicle type on junction capacity and delay will be investigated and analysed in terms of passenger car unit values at different highway junctions controlled by traffic signals.

Adams (43)and Kinzer (44)first investigated the free flowing conditions of the traffic flow. It was shown that in these conditions the arrival distribution of vehicles could be approximated by the Poisson distribution.

The application of the Poisson distribution to traffic was studied by Pak-Roy (45). The data which was obtained from the four sites in Australia was divided into 30 second intervals and the frequency of arrival was noted and compared with the frequency obtained from the Poisson distribution.

The class interval was one second,and a cumulative frequency curve was fitted to the theoretical exponential curve and plotted on semi-log paper for the volumes observed. From these results and graphs, it was concluded that the upper limit to the volume at which vehicles will be distributed at random varied with the location. In each case the critical range of volume, at and above which the Poisson distribution ceased to a good approximation, agreed with the practical capacity of the facility being considered.

Traffic flow rates were described by Buckley (46) to vary as a function of time. Buckley proposed a semi-random headway distribution for a free flowing single lane traffic flow in which it was assumed that behind each vehicle there is a zone which vehicles never enter. It was found that the negative exponential distribution was an extremely poor fit to the observed data, and both the generalised Pearson type III distribution and the semi-random distribution gave an acceptable fit to a limited amount of very high volume free-headway frequency data. A modified binomial distribution was suggested by Lewis (47) in which two different levels of probability were employed. The following equations were developed:

$$Pa = 1 - (1-B)^{1/R-e+1}$$

and

$$Pb = (1-B) / \bar{h} - e - \sum_{n-1}^{R-e} (1-Pa)^{n}$$

where

Pa = enhanced probability of an arrival at a time increment when $e \leq h \leq R$

Pb = diminished probability of an arrival at a time increment when $h > R$

B = bunching factor which is the fraction of all headways $\leq R$

R = maximum headway for which the probability of an arrival is enhanced.

e = minimum headway permitted

$\bar{h}$ = mean of all headways, and

h = any value of h.

Lewis proposed 0.5 seconds for "e", 4.5 seconds for "R" and $(1-e^{-0.00132V})$ for B.

Schull (48) proposed that when traffic suffers some degree of congestion, the headway distribution could be considered to be composed of two exponential curves, one representing those drivers who were unable to overtake and were restrained in their driving performance and the other representing those drivers who were unrestrained by other vehicles on the highway. Schull suggested that if "L" is the fraction of total volume made up from constrained vehicles and (1-L) is the fraction of total of volume made up of free-flowing vehicles,

$$P1,2 = L \exp\left(- (t-E) / (t1-E)\right) + (1-L) \exp\left({}^{-t}/t2\right)$$

in which

P1,2 = Probability of arrival from a composite distribution of restrained and unrestrained;

t1 = average time spacing of restrained vehicles.

t2 = average time spacing of unrestrained vehicles.

E = the minimum time headway between following vehicles.

Salter (49) investigated the parameters of Schull's formula and a regression analysis has been carried out to obtain relationships between the traffic volume and L, t1, t2. In the study three types of highways were included and the relationships between traffic volume and the proportion of restrained vehicles L were investigated by:-

2.2 - Spacing and headway characteristics.

The distance between successive vehicles travelling in the same direction on a highway and the interval of time between arrival of successive vehicles at a fixed point represent fundamental parameters in the study of the traffic flow.

Capacity studies of intersections, weaving areas, ramps and other analyses of roadway characteristics, have required the investigation of spacing and headway characteristics. Vehicular spacing also has its application in predicting arrival rate at a point, testing the randomness of traffic flow and estimating gaps and delays at vehicular and pedestrian crossings. Because the inverse of the mean time headway is the rate of flow, headways have been described as one of the fundamental building blocks of traffic flow. When the traffic flow reaches its maximum value then the time headway reaches its minimum value and under very heavily trafficked conditions, all vehicles are travelling at uniform headways as they follow each other along the carriageway.

There are several probablistic models which represent the distribution of the arrival time intervals. The most widely used are the negative exponential distribution, the shifted exponential distribution and the double exponential distribution.

a - a linear relationship with the proportion of restrained drivers as the dependent variable

b - an exponential relationship of the form

$L = 1-(\exp(A-B \text{ Volume})) / 100$

The following relationships were obtained:-

(A) one-way highways with two traffic lanes

a - $L = 0.00158. \text{ Volume} - 1.04222$

$1295 > \text{Volume} > 660$

$R = 0.41$

b - $L = 1 - (\exp(6.95042 - 0.00321.\text{Volume})) / 100$

$\text{Volume} < 730$

$R = 0.37$

(B) two lane two-way highways

a - $L = 0.00146. \text{ Volume} - 0.52985$

$790 > \text{Volume} > 380$

b - $L = 1 - (\exp(5.56093 - 0.00238.\text{Volume})) / 100$

$\text{Volume} > 400$

(C) two lane two-way highways approximately ½ mile downstream of traffic control signals.

a - $L = 0.00122. \text{ Volume} - 0.71324$

$1020 > \text{Volume} > 580$

b - $L = 1 - (\exp(5.80784 - 0.001900.\text{Volume})) / 100$

For each type of highway the relationship between traffic volume and the mean headway of restrained vehicles was obtained:

Type A highway	2.51 sec
" B "	2.52 sec
" C "	2.58 sec

Gerlough (50) suggested that when vehicles were flowing in platoons or were restrained so that they could not pass at will, then the probability of a gap between successive vehicles of less than (τ) is zero. This phenomenon may be represented by an exponential curve shifted to the right by an amount (τ) or,

$$P(t) = 1 - \exp(-(t-\tau) / (E-\tau))$$

solving for t gives

$$t = \tau - (E-\tau) \log_e (1-P)$$

where

t = time headway

τ = the minimum headway

$(1-P)$ = the random number fraction

E = the mean time headway

Kimber, McDonald and Hounsell (51) indicated that the type of headway distribution characterizing the departure process should be noted first. They showed typical distribution for light and heavy vehicles headways shown in figure (2.2 -1). Normal and log normal distributions were superimposed with means and variances equal to those of the samples. The normal distributions were assumed for their research for convenience.

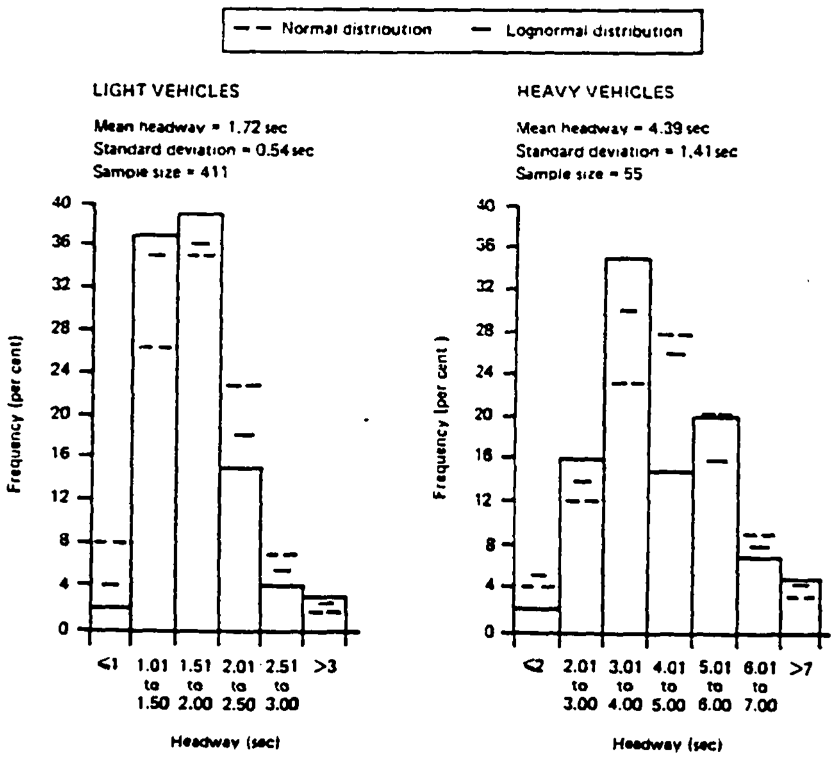


Figure 2.2.1 - Typical headway distributions for light and heavy vehicles. (Reproduced from reference No.51)

2.3 - Saturation flow criteria

2.3.1 - Definition and notation.

It is a well-known phonomenon that when the traffic signals turn green on an approach gaining right of way, the flow across the stop line quickly reaches a maximum steady value, the saturation flow, S, of the approach being expressed as equivalent passenger cars. Figure (2.3.1-1) shows, in idealized form, the variation of the flow of vehicles across the stop line of a traffic signal approach for which the queue waiting on the approach does not clear by the end of the green period. The flow remains at about the steady average value until the signal changes to amber, when it falls to zero during this period. The height of the rectangle is known as the saturation flow. The flow is often found to remain at this value until either the queue of vehicles waiting to pass through the approach is exhausted or the lights change to amber, which ever occurs sooner.

Where traffic signals are used to control a busy intersection for reasons of safety and efficiency, it is important that each road is given a suitable share of green time. If this is not achieved then one or more approach may experience excessive delay and drivers will become delayed and frustrated.

The design of intersections and setting of

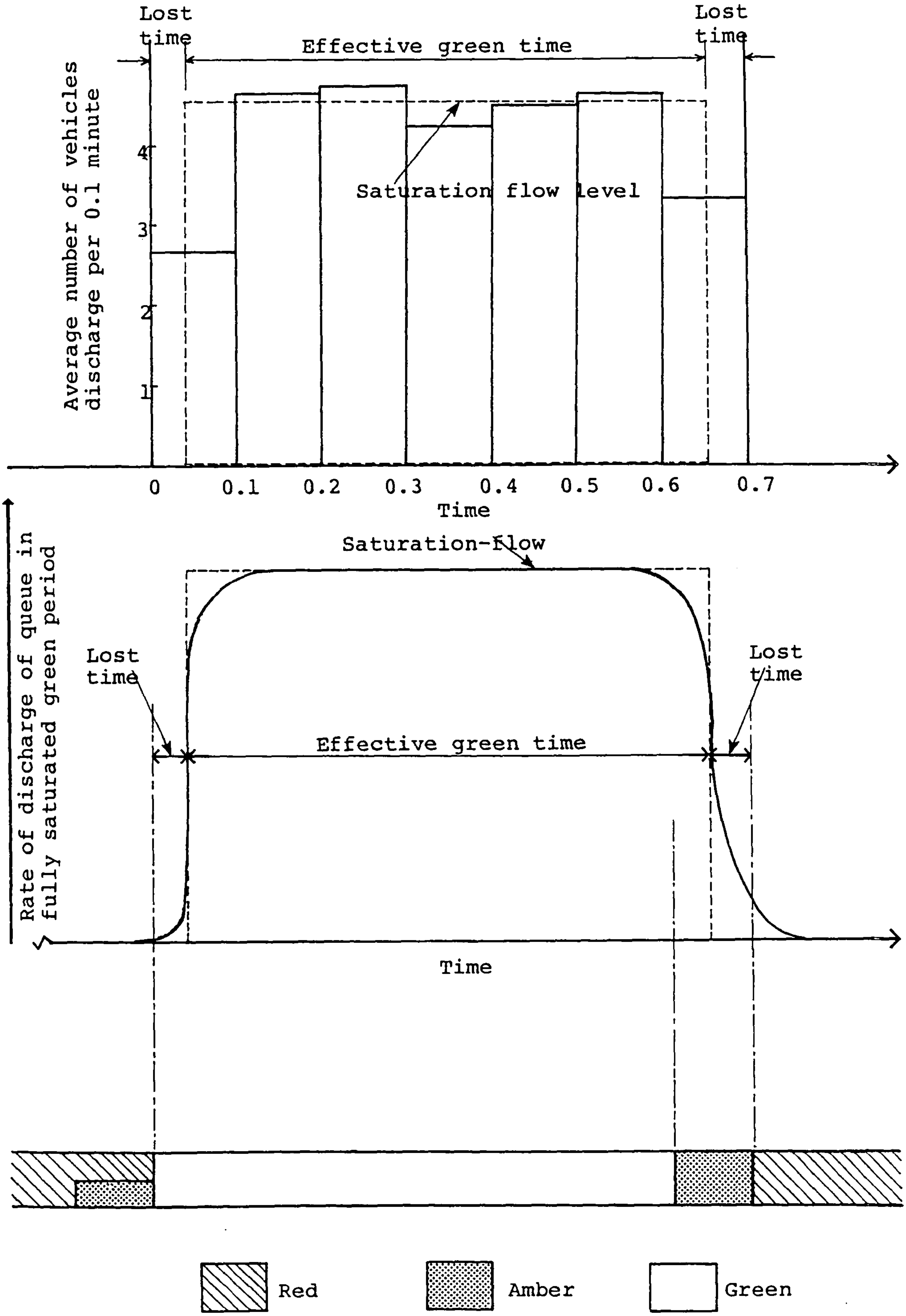


Figure 2.3.1-1 A typical example of signal aspect showing the variation of average discharge rate during a fully saturated green period.

signals is based largely on the volume of traffic involved and the need to safely separate the major conflicting movements in time.

A key factor in the determination of optimum layouts and signal settings is the maximum flow that can be accommodated by each arm or phase at the intersection. This maximum flow or 'saturation flow', can be measured directly in some situations when the intersection already exists, or more commonly, it has to be estimated from relationships based on geometric and other characteristics of the site. Thus in order to allocate green time efficiently, it is necessary to be able to condense the road widths and vehicle type flows into a single measure - the degree of saturation, by which approaches can be directly compared and their needs balanced.

2.3.2 - Effective green time.

The "effective green time" is the green time, G, minus the time lost at the beginning of green when vehicles are still accelerating, plus the time gained by vehicles making use of the amber period.

$$G_e = G - \lambda_1 + \lambda_2 \qquad \text{..........2.3.2-1}$$

where

G_e = effective green time

G = the green time

λ_1 = time lost at the beginning of the green time

λ_2 = time gained by vehicles making use of amber period

see figure (2.3.2-2)

The area under the curve in figure (2.3.2-1) represents the number of vehicles discharged during the period as mentioned before and, if this number is divided by the saturation flow, the resulting value is the effective green time. This is obviously less than the green time plus amber time, i.e. the area under the curve has been replaced by a rectangle of equal area, assuming the total passage of vehicles remains the same but that they flow at a constant rate during the effective green period.

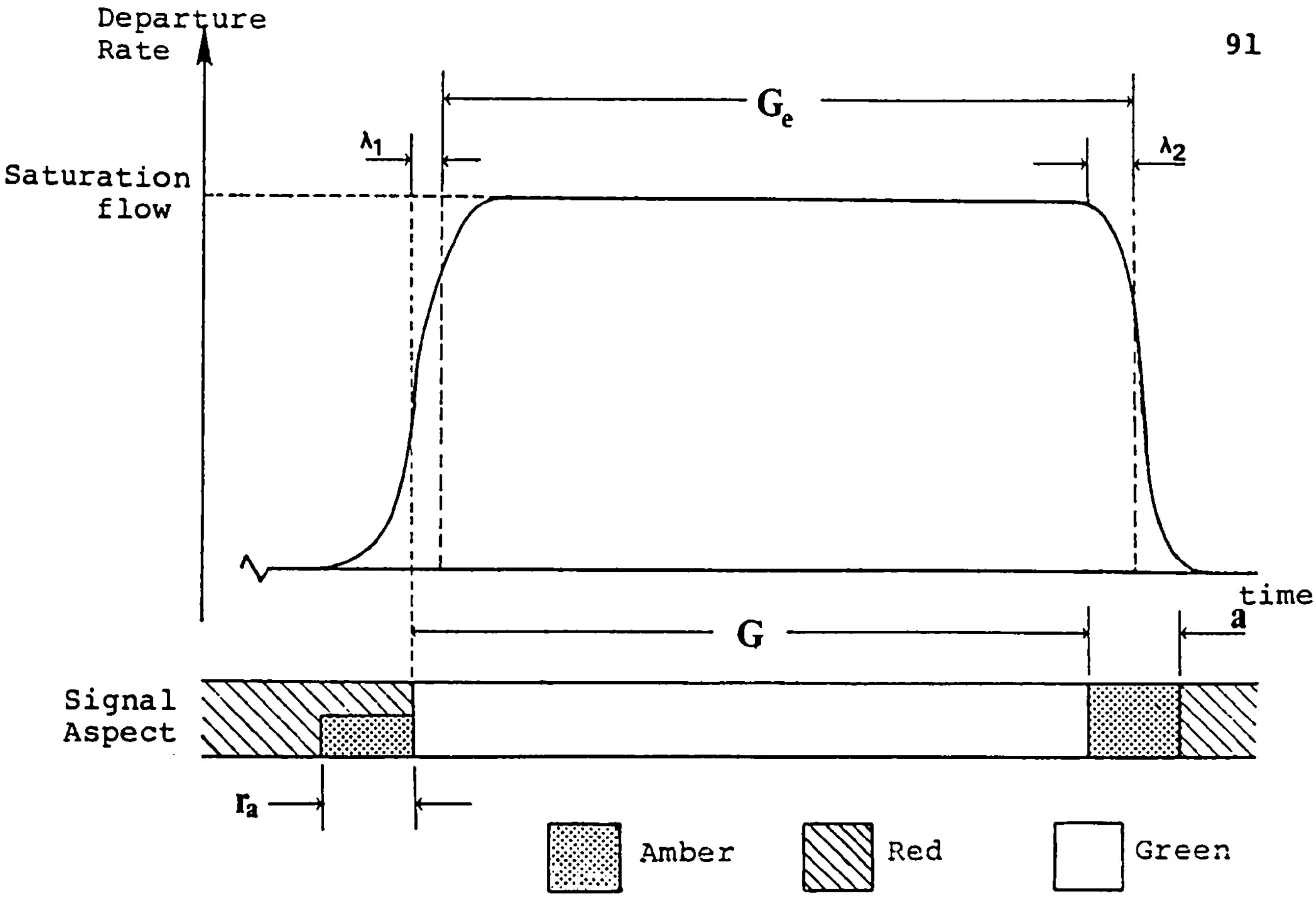


Figure 2.3.2-1 Parameters of the Traffic Signal Departure Process and Definition of Green Start Lag, λ_1 , and Green End Lag, λ_2

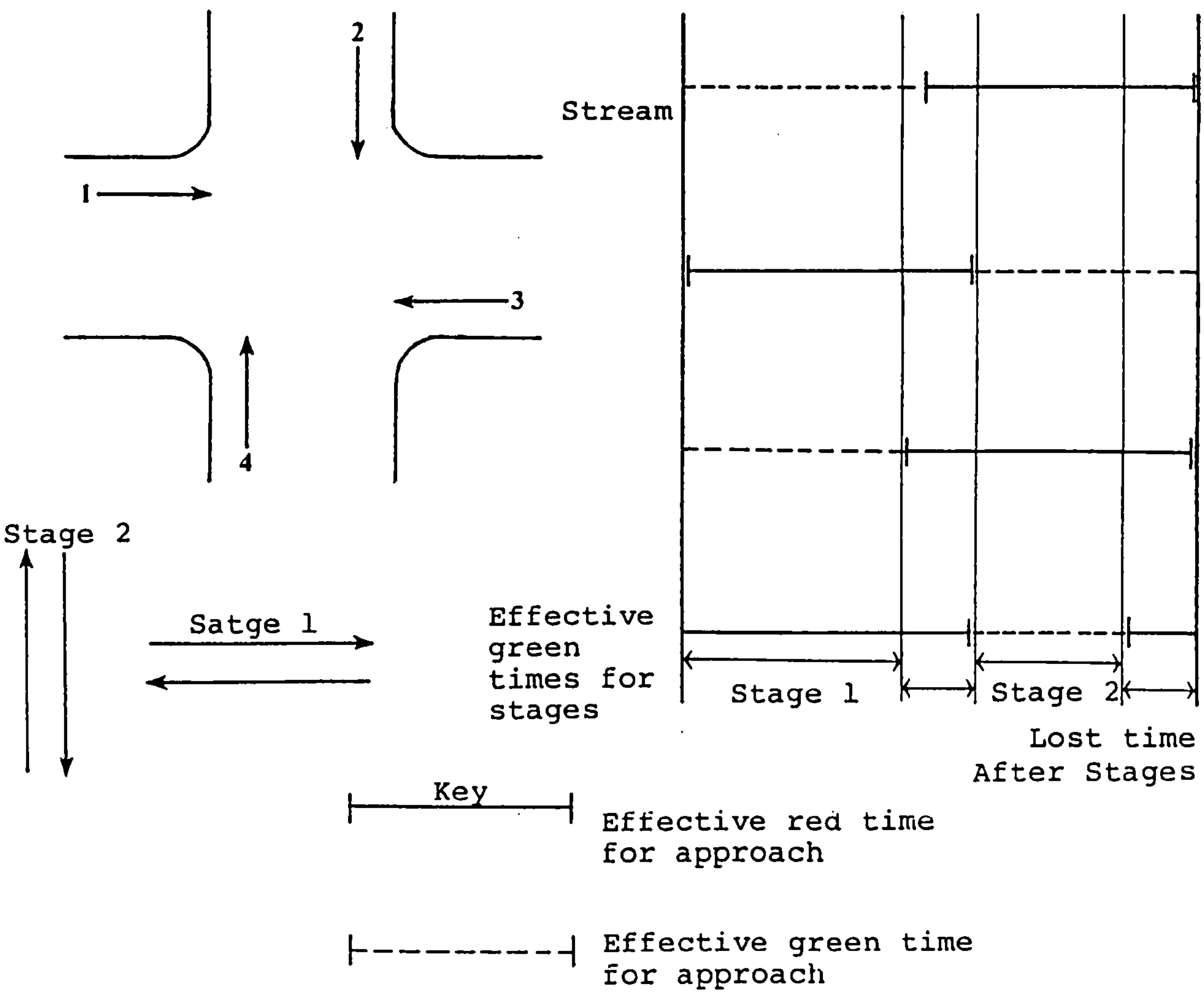


Figure 2.3.2-2 Showing sequence of stages and representation of effective green times and lost times.

The "lost time", L, between two successive stages is defined as the time between the end of the effective green time of the last approach losing right of way and the start of the effective green time of the first approach gaining right of way. This time is known as the "inter-green" time, "I", (i.e. the time from the end of the green period of the stage losing right of way to the beginning of the green period gaining right of way), minus the green end lag of the last approach losing right of way, plus the green start lag of the first approach gaining right of way, i.e.

$$L = I - \lambda_2 + \lambda_1 \qquad \ldots\ldots\ldots\ldots 2.3.2\text{-}2$$

Allsop (32) has defined the effective green time for each stage in the cycle as the time that is effectively green for every approach that has right of way in that stage and the lost time following each stage as the time from the end of its effective green period to the beginning of the effective green period of the next stage. Figure 2.3.2-2 illustrates the concepts of effective green time for a stage and the lost time following each stage for a simple cross-roads with two stages.

Although the length of lost times varied between sites, Allsop noted that the range of this variation was only between 2.9 to 3.5 seconds. The mean green end lag of 3.36 found by Miller (52) was very close to the mean length of lost time of 3.16 secs.

2.3.3 - Approach width and saturation flow relationship.

The relationship between saturation flow and approach width was established by Webster at the Transport and Road Research Laboratory during the mid 1950s and was published in 1958 (53). Observations of traffic flow were made by the Transport and Road Research Laboratory at about 100 signal-controlled junctions mainly in the London area and other locations in other large cities (25), supplemented by controlled experiments at the Transport and Road Research Laboratory test track. Their traffic experiments were carried out using different approach widths, up to a maximum of 60ft (18.3m), and varying arrangements of vehicle width in the "queue". The results have shown that the saturation flows, S, expressed in passenger car units per hour with no parked vehicles is given by;

$$S = 525\ W \quad \text{p.c.u/h} \quad (w>5\text{m})$$

(for values of W between 5.5-18m)

which is known as the Webster formula for the saturation flow, S, for the whole stop line, where W is the width of the approach in meters. For approach width less than 5 meters the corresponding saturation flow can be estimated from the following:-

Approach width W(m)	3.00	3.50	4.00	4.50	5.00
Saturation flow S pcu/h	1840	1885	1960	2210	2575

Branston (54) investigated the criterion of saturation flow with total effective width at four sites and at all times of day. The results were compared with the values recommended by Webster and Cobbe for sites classified as "good", "average" and "poor". Figure (2.3.3-1) shows the estimated saturation flows tend to be between the values recommended for different categories of sites. For single-sites the estimated saturation flows were mostly below "average", whilst those for two-lane sites were slightly above average. For this reason alternative formulae were investigated.

Saturation flow was found by Miller (52) to increase rapidly with increasing lane widths for lanes up to 3.05m wide, but that thereafter lane width had little effect. The variation of saturation flow "S" with lane width, "W" for the sites investigated by Branston is shown in figure (2.3.3-2). Three parallel straight lines were suggested to give a good representation of the observed variation in saturation flow at different times of day.

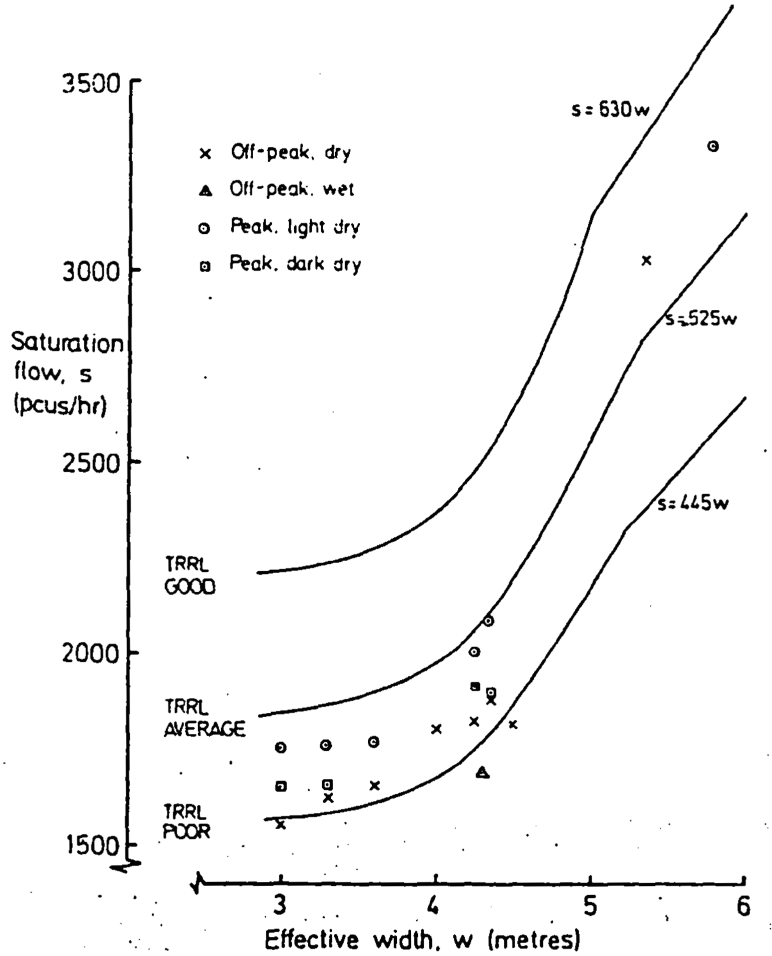


Figure 2.3.3.1 - The variation of saturation flow with overall effective width.
(Reproduced from reference No.54)

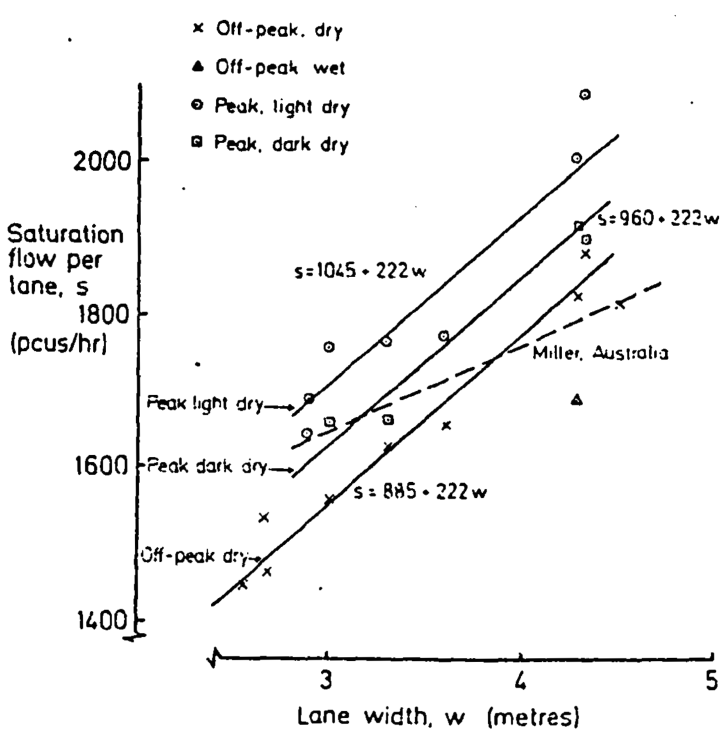


Figure 2.3.3.2 - The variation of saturation flow per lane with lane width.
(Reproduced from reference No.52)

It was found also that saturation flow dependence on lane width is much stronger than that found by Miller or by Leong (55), who found that lane widths in the range 2.75m to 3.5m had little effect on saturation flow.

The Transport and Road Research Laboratory (56) carried out a full scale traffic experiment to investigate the performance of conventional and new traffic signal junctions. The relationship between saturation flow per lane, s and lane width, W, was summarised by the equation:

$$s = 196W^2 - 979W + 2964 \text{ pcu/h } (2.5\text{m} \leq W_1 \leq 4\text{m})$$

and the saturation flow for the whole stopline was

$$S = \Sigma s = ns$$

where n is the number of lanes.

The comparison of their results with that of Webster's could be conducted strictly for a given lane width (i.e. for $W = 3.0\text{m}$ $S = 597W$) which was quite close to Webster's results.

It was found also that there was no indication of any substantial change in mean saturation flow values over the last 20 years or so. They suggested that the reason for that was the factors limiting the saturation flow were primarily determined by driver response and did not depend on vehicle performance, as the latter was improved during this period. The other reason was the effect of heavy vehicles present in the traffic system in recent years.

The Transport and Road Research Laboratory 1986 (57) latest report, suggested that saturation flow for a non-nearside lane of 'average' width, 3.2m, is now around 2080 PCU/h, some 15 per cent or so higher than the equivalent implied by Webster's 1966 value of around 1800 PCU/h. For nearside lanes, the present figure is about 1940 PCU/h, an increase of some 8 per cent over the Webster's figure.

It was also suggested that for individual lanes containing straight-ahead traffic, saturation flows are decreased by 2 per cent per one per cent of gradient uphill, but were unaffected by downhill gradient. They are increased with lane width by 100 PCU/h per meter of road width and were lower by 6 per cent in wet road conditions than in dry. In nearside lanes they were lower by about 140 PCU/h than in other lanes.

The change in the saturation flows prediction is due to the change of a number of things such as vehicle performance and road markings and layout practice.

2.3.4 - Methods of estimation of saturation flow.

Saturation flow was estimated by the Transport and Road Research Laboratory in Road Note 34, (1963) (58) by recording the number of vehicles passing the stop line during successive short intervals of time when the signal approach is saturated, i.e. while there is a queue of vehicles waiting to pass through. Flows in intervals which are free from start and end lag effects are averaged to give a measure of saturation flow. In this method saturation flow is expressed in vehicles per unit time and does not allow PCU values of different vehicle types to be calculated from observed data.

The headway method is the alternative to the counting method of the TRRL. The inter-arrival times of all saturated vehicles are measured at the stop line. The headways of successive saturated vehicles were found to be constant after the fifth (59) or the fourth (55) in line. Saturation flows are calculated directly in PCUs/unit time as the reciprocal of the average headway of saturated straight-on passenger cars. The PCU values of different vehicle types are obtained by comparing the headways of these vehicle types with those of straight ahead passenger cars.

Miller (52) has compared the two methods and found that when several vehicle types were being recorded, the TRRL counting method requires too much information to be written down by observers, so cassette recorders were thus used instead to record the information. Miller also found that in the headway method, the analysis period was extremely long, and its use would not be recommended for large scale data collection.

More recently large scale empirical studies have been undertaken to extend and update the methods for the prediction of saturation flows and lost time at signal controlled intersections. In a recent study (60), undertaken by the consultants Martin and Voorhees Associates (MV) for the Transport and Road Research Laboratory, saturation flows were recorded for individual lanes at some 37 sites through the U.K. The traditional 6 seconds time intervals, vehicle count technique as described in Road Note No. 34, were used as the basis for recording flows during the surveys. The lane configurations samples including straight ahead only, left turn only, or right turn only, (opposed and unopposed) movements. Relationships were obtained using regression techniques for prediction of saturation flow for these turning movements based on readily measurable parameters such as curvature and lane position.

Branston and Van Zuylen (61) suggested a method to estimate saturation flow using multiple linear regression. Green and amber time was divided into three consecutive counting periods, first, middle and last, see

Figure (2.3.4-1). The end of the last counting period of fully saturated cycles was fixed at the change to an amber light, but the ends of the first and middle counting periods might be chosen either:

i) to correspond exactly with the instant of departure of a specified vehicle, which was referred to as "Synchronous" or

ii) as an arbitrary point in time which was referred to as "Asynchronous".

Typical counting periods for the two methods of counting were superimposed on an idealised model of the departure process as shown in figure (2.3.4-2). The green start and end lags implied by synchronous method were not the same as those implied by the asynchronous method. Van Zuylen developed the first method using synchronous counting. A double dial stop watch was used by the observer who varied the number of vehicles in each counting period and recorded the number and types of vehicles departing in each period as well as the stopping times. The first counting periods begin at the instant the green light shows and ends at a time when the departure rate is the saturation flow (vehicles departing before the green light are included in this period); the middle counting period ends at a time when the departure rate is still the saturation flow and the last counting period ends when the amber light shows (vehicle departures occuring during amber and after the red light are included in this period).

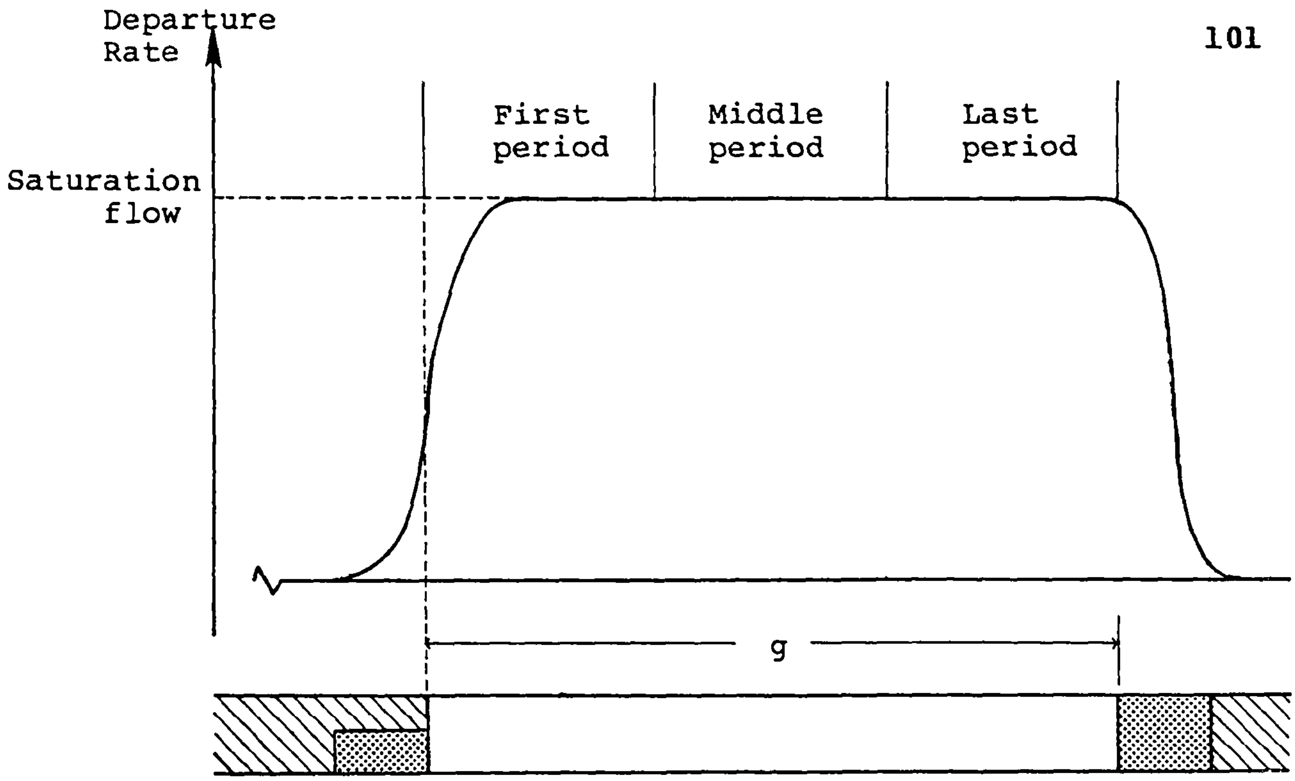


Figure 2.3.4-1 Division of the green time into first, middle and last counting periods.

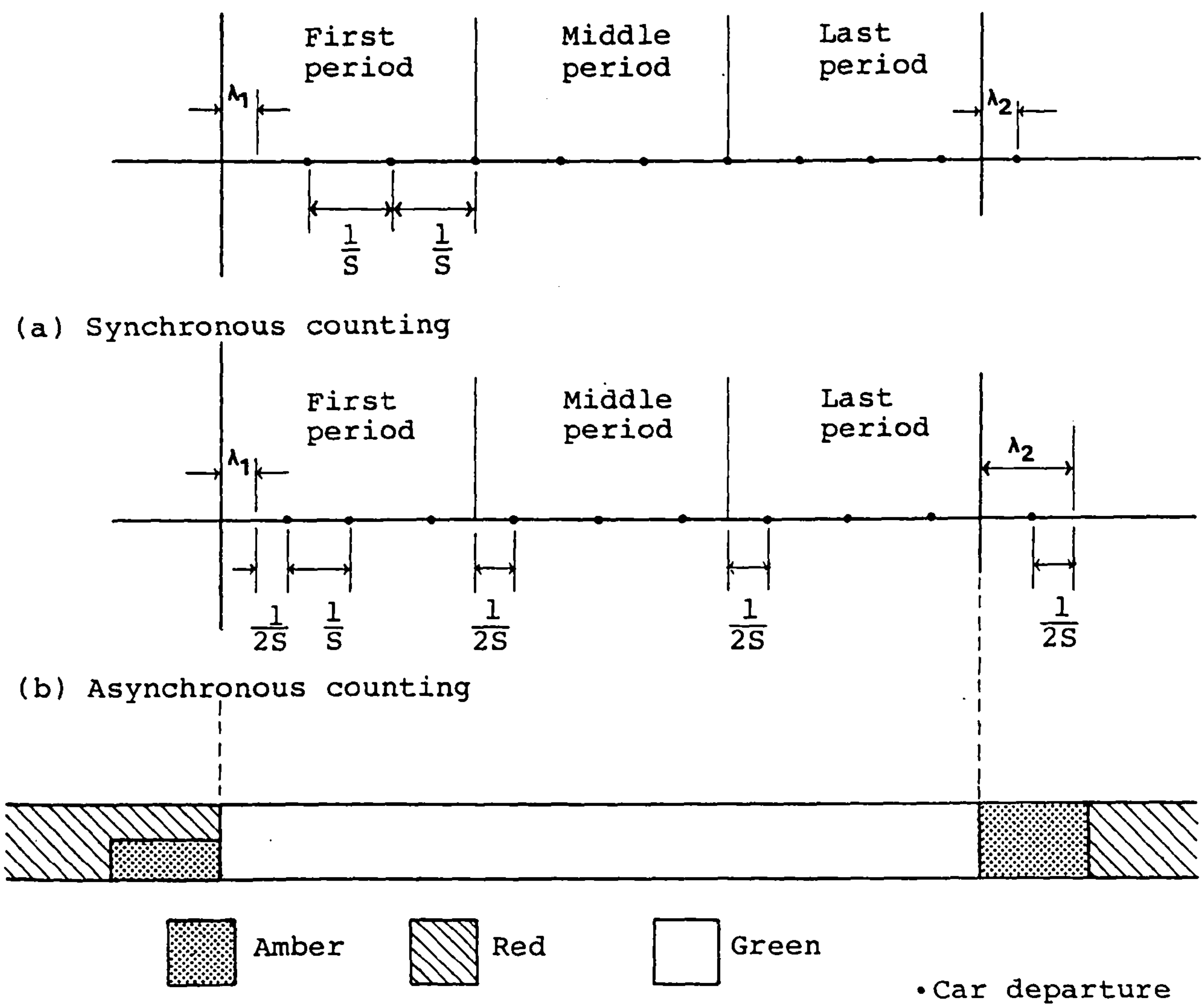


Figure 2.3.4-2 Definition of synchronous and asynchronous counting methods.

2.4 - Current approach to the use of passenger car units

2.4.1 - Definition.

The passenger car unit equivalent of vehicle is the number of passenger cars which would have the same effect on capacity as the vehicle being considered under the given roadway conditions. PCU values represent the effects of changes in traffic composition (the mix of cars, goods vehicles, buses, ... etc.) on the saturation flows at traffic signals as well as at other roadway junctions. Each type of vehicle in the traffic stream is equivalent to a number of passenger cars or private cars in respect to its road-capacity requirements. The capacity of a road is traditionally expressed as the maximum number of vehicle per unit of time that can flow past a point. The utilization of a road can be expressed as the product of flow and time or vehicle-hours. One vehicle-hour of utilization could be a solitary vehicle travelling for one hour or a stream of 60 vehicles travelling for one minute; in both cases the product of flow and time is one and this is the measure of the utilization of that road. PCU values are, by definition, measured relative to the passenger car as the base vehicle. Different types of vehicles, due to variation in their performance, use differing amounts of vehicle-hours. Heavy goods vehicles for example, which may be longer and slower than the average vehicle, require a greater number of vehicle-hours to make the same trip as a private car. The introduction of a larger or slower vehicle into a stream of traffic also causes other vehicles to slow down and use extra vehicle-hours.

2.4.2 - Methods of Calculating PCU's.

Different methods of estimating PCU's values have been developed by many researchers at different highway intersections. The main methods widely used are the following;

i) Webster's method.

In a controlled "track" experiment performed by Webster (1958) (53), and Charlesworth and Webster (1958) (62), vehicles were classified as "light ", which include cars, taxis and light commercial vehicles and as "goods vehicles" which included medium and heavy commercial vehicles and some double deck buses. In the various classes, the numbers of vehicles were adjusted progressively throughout a continuous test so that the percentage of goods vehicle ranged from 0 to 100 per cent.

Vehicle departures were registered at the stop line using paper tape punches and employing event-recording techniques. The PCU value of a goods vehicle was determined by grouping data for successive sets of about 12 signal cycles and plotting for each set the average number of goods vehicles per cycle

$\bar{n}_g = \Sigma n_g / N$ against the number of light vehicles per cycle $\bar{n} = \Sigma n / N$, where n_g and n are the numbers of departing goods vehicles and light vehicles in individual cycles and N in the number of cycles in the set. There was very little scatter and a straight line was drawn through $\bar{n}g$, $\bar{n}l$ values by eye. The PCU value was estimated as $-1/\text{slope}$.

ii) Car following method.

The behaviour of a single-lane traffic stream can be analysed by examining the manner in which individual vehicles follow one another and form the behaviour of pairs of vehicles. The passenger car equivalent of a commercial vehicle may be determined by noting the time headway between successive vehicles as they cross the stop line at a saturated traffic signal approach. The traffic is confined to a single lane and all vehicles continue straight across the intersection. The observed headways are then classified into the following:-

(1) Passenger car following passenger car

(2) Passenger car following commercial vehicle

(3) Commercial vehicle following a passenger car

(4) Commercial vehicle following a commercial vehicle

By dividing the mean headway for a commercial vehicle following a commercial vehicle by the mean headway for a passenger car following a passenger car the PCU for a commercial vehicle can be found. This statement is only true if the effect of a commercial vehicle is independent of whether the vehicles preceding it and following are light or heavy. One condition should be met too before the PCU for a commercial vehicle could be determined;

that the sum of the average headways for a passenger car following a passenger car and a commercial vehicle following a commercial vehicle is equal to the sum of the average headways for a passenger car following a commercial vehicle and a commercial vehicle following a passenger car. Without this condition, corrected values of the mean headways can be calculated as follows:-

Corrected value of mean headway for a passenger car following a passenger car = Uncorrected value (w) - correction/number of headways of this type (a).
Corrected value of mean headway for a passenger car following a commercial vehicle = Uncorrected value (x) + correction/number of headways of this type (b).
Corrected value of mean headway for a commercial vehicle following a passenger car = Uncorrected value (y) + correction/number of headways for this type (c).
Corrected value of mean headway for a commercial vehicle following a commercial vehicle = Uncorrected value (z) - correction/number of headways of this type (d).
where

$$\text{Correction} = \frac{abcd\ (w-x-y+z)}{bcd+acd+abd+abc} \qquad \ldots\ldots\ 2.4.2\text{-}1$$

In a controlled experiment carried out by the Transport and Road Research Laboratory, using the car following method, Scraggs (63), found that for a

3.05m approach width at traffic signals, the passenger car equivalent of heavy vehicles was 1.47, and for 3.65m approach width a value of 1.68 was obtained.

iii) Regression methods.

The two categories used in the regression methods by Holroyd (64), Branston and Van Zuylen (61), Branston and Gipps (65) and Kimber et al (51), were the counting methods termed "Synchronous" and "Asynchronous" which were mentioned in paragraph (2.3.4). In the synchronous method, the number η_i of vehicles departures of each class i are recorded over time period t, beginning and ending with the departure of a vehicle (the first vehicle departing was excluded from n_i so that t contained n_i headways), "and t" was regressed on the n_i to obtain estimates a_i of the coefficients η_i in the linear model:

$$t = \sum_i \eta_i \; ni + \varepsilon \qquad \text{..........} \; 2.4.2\text{-}2$$

where ε is an error term.

The regression plane was constrained to include the origin, and the counting period was chosen to exclude start and end effects (lost time eifects). The coefficient η_i of the equation above, represents the population mean headway of vehicles of the class i, thus if i = 1 denotes passenger cars, a_ℓ estimates the mean car headway. Estimates of the PCU values could be obtained from the ratio a_i/a_ℓ.

In the asynchronous counting method the vehicle departures were recorded over periods T, which begin and end at arbitrary instants. Then the number of passenger cars n_ℓ was regressed on T and on the number of vehicles n_i $(i \neq 1)$ of other classes i to obtain estimates b_o, b_i of the coefficients β_o, β_i in the model:

$$n_\ell = \beta_o T - \sum_{i \neq 1} \beta_i n_i + \varepsilon \qquad \ldots\ldots\; 2.4.2\text{-}3$$

where β_o and β_i represent, respectively, the saturation flow in PCU per unit time and the PCU value of vehicles of class i. Equations of this type were discussed by Branston and Gipps (65) who have investigated the effects of non-constant variance in the counts with respect to time. They have demonstrated that constant variance can be achieved in error term by weighting by $1/\sqrt{T}$.

Thus

$$\frac{n_\ell}{\sqrt{T}} = \beta_o' \sqrt{T} - \sum_{i \neq 1} \frac{\beta_i'' n_i}{T} + \varepsilon' \qquad \ldots\ldots\; 2.4.2\text{-}4$$

The sampling variances of the estimates of the coefficients were reduced by using this model. Alternatively, the data could be subdivided so as to give the numbers of vehicle departures in time intervals of equal duration t, then a simpler model could be used in which the question of non-constant variance with respect to time does not arise:

$$n_\ell = \beta''_o - \sum_{i \neq 1} \beta''_i \; n_i + \varepsilon'' \quad \ldots\ldots\ldots \; 2.4.2\text{-}5$$

n_ℓ was regressed on n_i to obtain estimates b''_o, b''_i of β''_o and β''_i, which represent, respectively, the saturation flow in time t and the PCU value of vehicles of class i.

iv) Simulation method.

Kimber, McDonald and Hounsell (51), have carried out a research using simulation model based on synchronous and asynchronous vehicle counts using the regression analysis method in an attempt to obtain PCU values in saturation flows at traffic signal junctions. The relationship between the various methods of PCU derivation used previously by Webster's method and headway ration methods were found to agree as long as there was variability in the headways of vehicles of a given class (e.g. in car-to-car headways). The simulation was carried out based on the four equations previously mentioned in the Regression methods. Although saturation flows (in pcus/h) varied from site to site where data collection was designed to include different geometric layouts of sites chosen in the study, the pcu values were found to be relatively stable.

Table (2.4.2-1) summarizes the mean PCU values over all sites and the standard errors of these means arising out of total site-to-site variation.

Study	Method of derivation	PCU values standard errors are given in brackets Medium goods vehicle	Heavy goods vehicle	Buses
1	Asynchronous regression			
	t = 6 seconds	1.10 (0.02)	1.60 (0.05)	1.33 (0.04)
	t = 12 seconds	1.19 (0.04)	1.79 (0.07)	1.40 (0.09)
	t = 18 seconds	1.22 (0.04)	1.80 (0.10)	1.31 (0.10)
2	Headway ratio method	1.53 (0.05)	2.29 (0.14)	1.87 (0.10)
2	Asynchronous regression			
	t = 6 seconds	1.18 (0.05)	1.54 (0.11)	1.38 (0.12)
	t = 12 seconds	1.32 (0.07)	1.75 (0.20)	1.53 (0.15)
	t = 18 seconds	1.24 (0.13)	1.73 (0.15)	1.63 (0.19)
2	Synchronous regression	1.50 (0.07)	2.38 (0.18)	2.03 (0.16)

Table 2.4.2-1 Mean PCU values according to regression and headway ratio method.
(Reproduced from reference No. 51)

A macroscopic traffic simulation model was used in a research by Keller, 1984 (66) and Saklas, 1982 (67), on an urban network. This was used to derive passenger car equivalents estimates for large vehicles as a function of vehicle size, signal timing and traffic volumes. The simulation model called TRANSYT/7N is referred to as TRANSYT in the text. TRANSYT simulates a traffic stream rather than individual vehicle movements, as in microscopic simulation models. The program also uses seven visual vehicle classification types, six different levels of service, and three types of signal settings.

Three primary inputs were used in this estimation of total travel times from which the PCE values for different classes of vehicles were obtained. They were uniform delay, random delay and running time. Where uniform delay is defined as the entire amount of time spent by a vehicle not travelling at the free speed, which is computed by TRANSYT using

$$du = C. \sum_{t}^{N} \frac{m_t}{N} \qquad \ldots\ldots\ldots\ldots\ldots \; 2.4.2\text{-}6$$

in which du = uniform delay in veh-sec/cycle; C = cycle length in seconds; m_t = queue length during interval t in vehicles; and N = number of steps in the cycle.

Using time headways between different vehicles and the base vehicle, the weighting factor for adjusting uniform delay for different vehicle characteristics were obtained. Table (2.4.2-2) gives these factors.

Vehicle Type	Length ft (m)	Velocity m/sec	Time headway sec	Wdu
Auto	17 (5.18)	14.30	2.362	1.00
Bus	37 (11.28)	12.53	2.900	1.228
SU-2Ax *	24 (7.32)	12.95	2.565	1.086
SU-3An *	29 (8.84)	12.10	2.732	1.157
2S-1 *	35 (10.67)	12.95	2.824	1.196
2S-2 *	43 (13.11)	12.95	3.012	1.275
3S-2 *	52 (15.85)	12.62	3.265	1.382

Table 2.4.2-2 Weighting Factors for Uniform Delay.

* SU = Single unit; Ax = Axle; S = trucks combinations.
(Reproduced from reference No.66)

The second input was the random delay which could be expressed as

$$dr = \frac{15T}{S}\left(q-s+((q-s)^2+240\,\frac{q}{T})^{\frac{1}{2}}\right) \quad \ldots\ldots\ 2.4.2\text{-}7$$

in which dr = random delay in vehicle-sec/cycle; q = traffic flow in veh/sec/cycle; S = saturation flow rate in veh/sec of green; and T = simulation time-length of time for which stated network traffic conditions exist in minutes.

Weighting factors were developed to reflect the effect of vehicle performance on random delay which is summarized in table (2.4.2-3).

Vehicle type	Length ft (m)	Maximum acceleration	Wdr
Auto	17 (5.18)	6.6	1.00
Bus	37 (11.28)	5.9	1.229
SU-2Ax	34 (7.32)	5.7	1.140
SU-3Ax	29 (8.84)	4.4	1.348
2S-1	35 (10.67)	4.7	1.359
25-2	43 (13.11)	4.1	1.531
35-3	52 (15.85)	4.1	1.612

Table 2.4.2-3 Weighting Factors for Random Delay. (Reproduced from reference No.66)

The third input was the travel time for the unconstrained flow on each link i.e the flow that is free of any vehicle interaction. Table (2.4.2-4) presents the mean values of the characteristics of each vehicle type and the respective computed link travel times.

Vehicle type	Vehicle length ft (m)	Travel time sec.
Auto	17 (5.18)	35
Bus	37 (11.28)	40
SU-2Ax	24 (7.32)	39
SU-3Ax	29 (8.84)	42
2S-1	35 (10.67)	40
2S-2	43 (13.11)	40
3S-2	52 (15.85)	42

Table 2.4.2-4 Link Travel Times by Vehicle Type. (Reproduced from reference No.66)

The PCE values were calculated over a wide range of flow rates to approximately simulate levels of service from A to F. Two different sets of values were used for the percentage of each vehicle type in the traffic stream shown in table (2.4.2-5).

Vehicle Type	Percentage in vehicle mix	
	Peak	off-peak
Auto	89.0	83.9
Bus	0.9	1.3
SU-2Ax, 6-tire	2.7	4.0
SU-3Ax	0.7	1.0
2S-1	0.3	0.5
2S-2	2.2	3.2
3S-2	3.5	5.1
Recreational vehicle (not used)	0.7	1.0

Table 2.4.2-5 Composition of Typical Vehicle Mix. (Reproduced from reference No.66)

The computer simulation output and the computed delay weighting factors provided all the information necessary to compute PCE values.

The PCE values for each vehicle type K are given by

$$PCE^K = \frac{TT^K}{TT^b}$$

in which PCE^K = the PCE value of the given vehicle type K

TT^K = the total travel time of vehicle type K over the network, in hours; and

TT^b = the total travel time of the base vehicle over the network in hours.

The computed PCE values (66) by vehicle class and traffic characteristics are summarized in table (2.4.2-6).

The model is easy to use because it requires a minimum amount of data collection.

Vehicle type	Signal* setting	Off Peak LOS A	LOS B	LOS C	LOS D	Peak LOS E	LOS F
Auto		1.00	1.00	1.00	1.00	1.00	1.00
Bus	a priori	1.16	1.17	1.17	1.16	1.16	1.22
	initial	1.14	1.14	1.14	1.14	1.14	1.22
	optimum	1.19	1.20	1.20	1.20	1.21	1.22
SU-2 Ax	a priori	1.11	1.11	1.11	1.11	1.11	1.13
6 tire	initial	1.03	1.08	1.09	1.09	1.09	1.13
	optimum	1.09	1.15	1.15	1.15	1.14	1.14
SU-3	a priori	1.20	1.19	1.20	1.19	1.20	1.30
	initial	1.17	1.17	1.17	1.16	1.16	1.30
	optimum	1.24	1.24	1.24	1.23	1.24	1.32
2S-1	a priori	1.16	1.15	1.15	1.16	1.17	1.30
	initial	1.14	1.13	1.14	1.14	1.16	1.31
	optimum	1.19	1.19	1.20	1.20	1.20	1.32
2S-2	a priori	1.23	1.17	1.18	1.18	1.19	1.44
	initial	1.21	1.15	1.16	1.16	1.18	1.45
	optimum	1.27	1.20	1.21	1.22	1.22	1.46
3S-2	a priori	1.23	1.23	1.25	1.25	1.26	1.53
	initial	1.21	1.21	1.22	1.22	1.25	1.53
	optimum	1.27	1.27	1.29	1.29	1.30	1.54

Table 2.4.2-6 Summary of Synthesized Passenger Car Equivalents.
Reproduced from reference No.66)

* Three different signal settings: a priori is a value chosen by the analyst and two TRANSYT generated values, the first an initial value and the second an optimum value.

2.5 - Site selection and description

2.5.1 - Introduction

In order to obtain a wide range of data to study traffic performance at signalized intersections, a number of sites were selected. At each of these sites, conditions of saturated flow existed for a period of forty minutes. The sites chosen were in Bradford, London and Leeds.

Data was collected from these sites at different times of the day (morning and afternoon), and on different days over a period during 1983, 1984 and 1985.

The results of the observation can therefore be considered to have a direct relationship to traffic flows at several busy urban intersections.

2.5.2 - Site selection criteria.

Sites were chosen so as to embrace as large a range of carriageway widths as possible, of recent design and construction. All the sites were subjected to heavy traffic flows. The sites chosen not only carried sufficient traffic to become saturated but also carried a range of different types of vehicle in order that PCU values could be simultaneously calculated. Furthermore they had to be exempt from outside influences, in that they could not be located in such a position that traffic from the adjacent junction blocked the observation or even reached back far enough to impede the flow of vehicles leaving the intersection. In addition bus stops and similar obstructions were not located immediately beyond the exit to the intersection. The chosen sites had also to be carefully selected so as to eliminate all other factors influencing traffic flow. In addition to the influencing factors already mentioned above the effect of gradient and other site characteristics had to be considered. Thus the sites chosen had to fulfil stringent requirements in order to be sure that these factors had been eliminated. These requirements were as follows:-

(a) The sites must cover a wide range of stop-line widths, including multi-lane approaches.

(b) The site must have a gradient of less than one percent, where gradient is measured as the average slope between the stop-line and a point on the approach about 60m (200 ft) before it.

(c) There were to be no parked vehicles or bus stops closer to the stop-line than the length of a normal maximum queue. Whilst this criterion necessarily involves subjective judgement, in general sites were chosen where no parking was allowed on the particular approach to be used.

(d) The site must have "average" site characteristics. As most characteristics are allowed elsewhere, visibility was chosen as the main criterion. Sites were to have good visibility with respect to their urban locations; those having particularly bad visibility or with an interrupted open aspect not giving completely clear views of the other traffic flows were eliminated. Any sites where there was a tendency for the flow to be measured to become blocked or affected by other traffic flows were also eliminated.

(e) Sites were chosen, whenever possible, where the pedestrian flows were few or non existent, but where this proved impossible, sites at which pedestrian flows were channelled to specific crossing points controlled by the traffic lights were chosen.

(f) Traffic flows at the site chosen should cover a reasonable variation in the mix of vehicle type.

(g) The approach should have sufficiently long periods of saturation flow.

(h) The sites involved should have a convenient location for filming the intersection.

Thus the selection of sites was based upon stringent conditions to ensure that the results obtained had logical and consistent basis and represented a wide range of different sites characteristics and different flow conditions.

Figure (2.5.2-1) illustrates the manner in which the differing aspects of the study are inter-related.

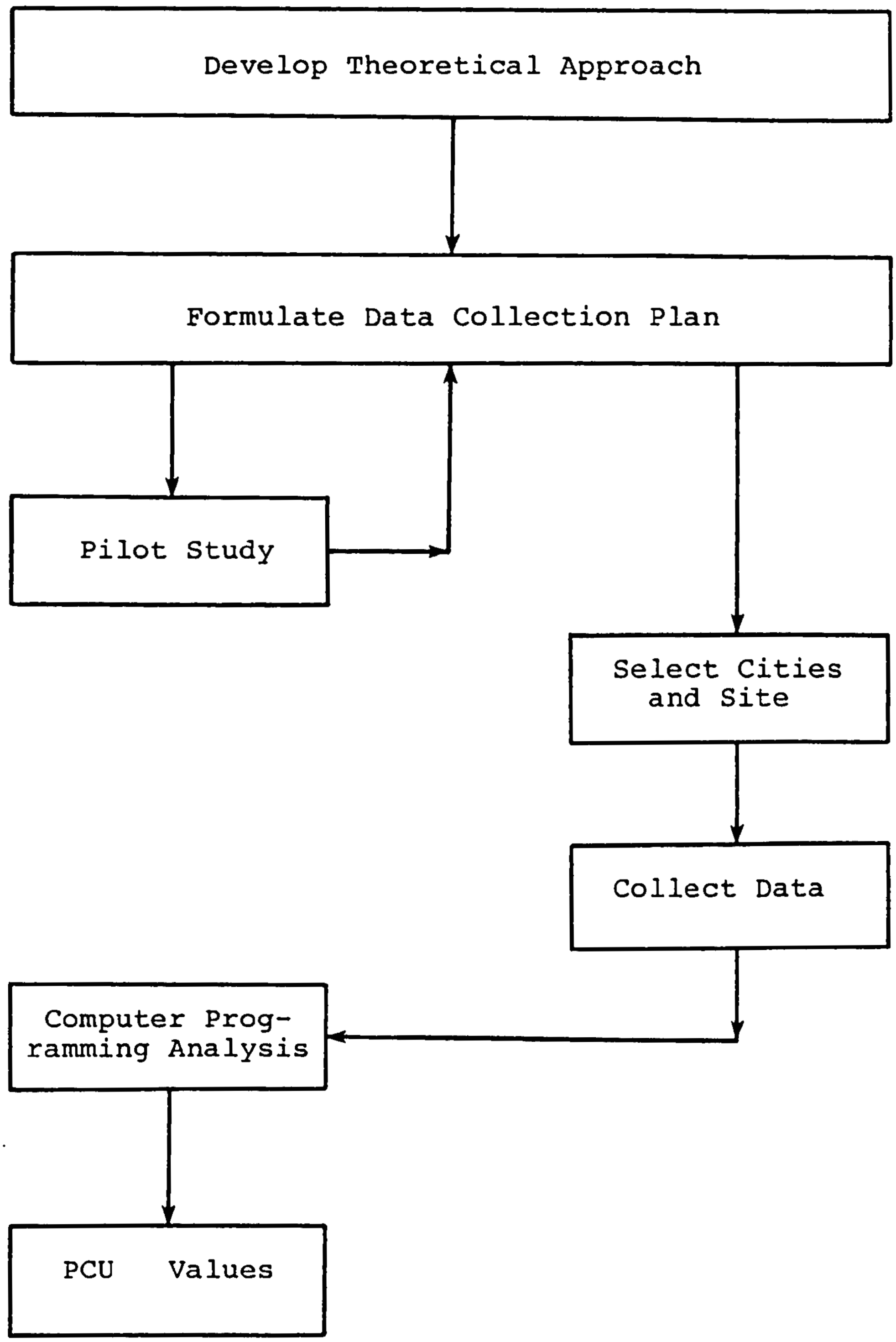


Figure 2.5.2-1 Study Outline

2.5.3 - Site descriptions.

Using the afore mentioned criteria the following sites were chosen:

(1) Thornton Road (B6145)/Whetley Lane (B6148) Signalized Intersection, Bradford.

This cross intersection is situated in the West Riding of Yorkshire, which is about a mile from Bradford City Centre (Fig. 2.5.3-1). For the study purposes Thornton Road approach was chosen where the traffic was travelling towards the city centre. There were two lanes at the approach, one for straight ahead and left turning traffic and the second an exclusive right-turning lane. The approach was almost level and straight for several hundred metres before the stop-line.

Left turning traffic was forced to negotiate a sharp turn. During morning peak hours the traffic queue towards the city centre was very long and the duration of saturation condition at this approach was approximately two hours between 7.30 a.m. to 9.30 a.m. The composition of traffic at the approach was 70% private passenger cars and the proportion of left-turning vehicles was 10% approximately.

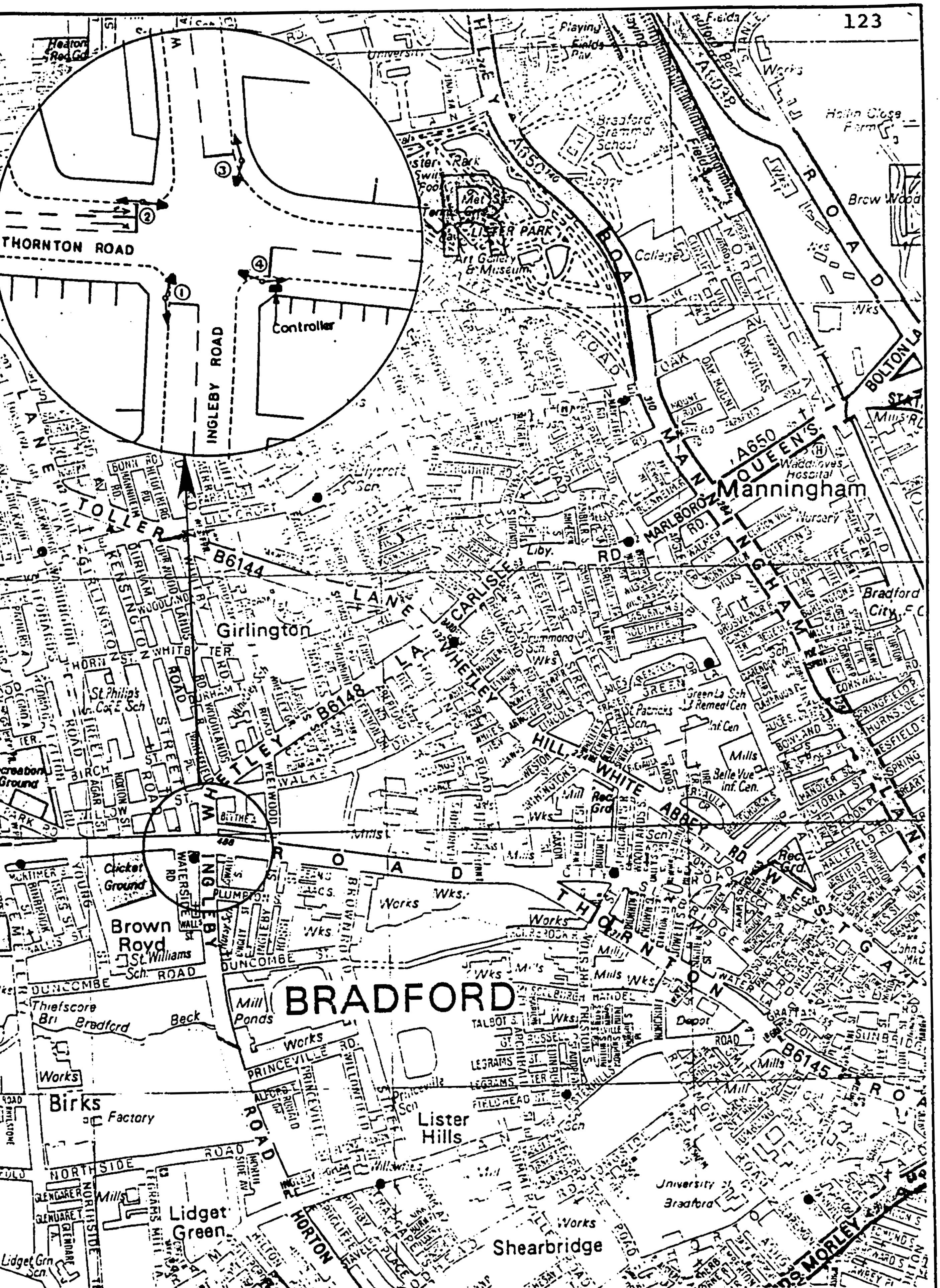


Figure 2.5.3.1 - Thornton Road/Ingleby Road signalized intersection (Bradford)

(2) Manningham Lane (A650)/Queens Road (A650) Signalized Intersection, Bradford.

This intersection represents an ideal situation for the investigation because of its location on a major route leading to Bradford City Centre (Fig. 2.5.3-2).

For the study purposes the approach on Manningham Lane was chosen where the traffic was leaving the city centre. The approach was almost level and had two lanes, one lane for straight-ahead and left-turning traffic and the other for right-turning vehicles only.

Saturation level was reached at the approach during afternoon periods between 2.30 p.m. and 5.30 p.m. At the afternoon peak times between 4.45 p.m. and 6.00 p.m. a high proportion of right-turning traffic existed, so a right-turning phase operated. Filming at this period of time was avoided.

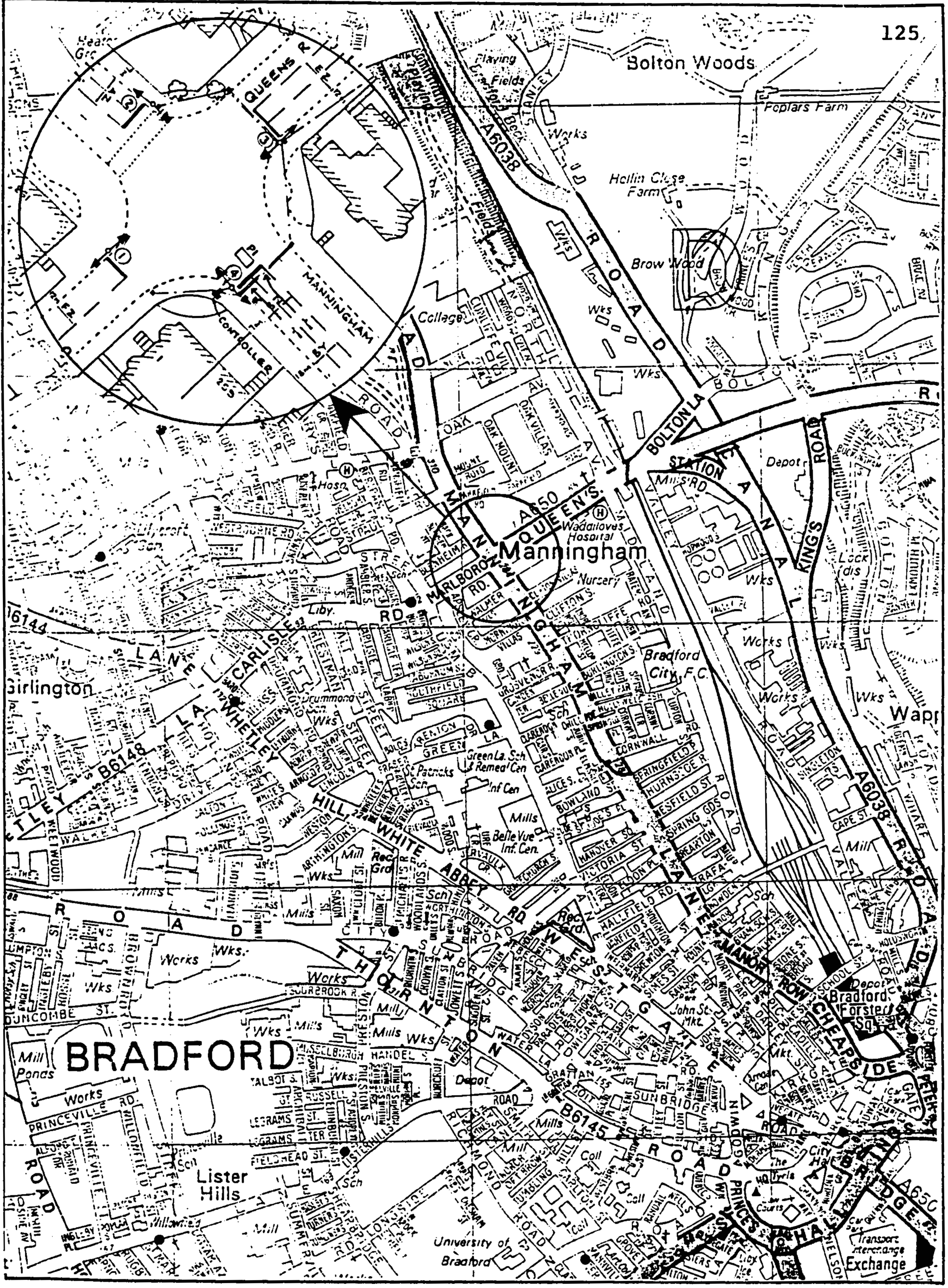


Figure 2.5.3-2 Manningham Lane/Queen's Road Signalized Intersection

(3) Woodhouse Lane (Hyde Park Corner) (A660)

Traffic Signal Approach, Leeds.

Woodhouse Lane represents one of the main routes leading to Leeds City Centre. The site chosen on this route was located at Hyde Park Corner (Fig. 2.5.3-3).

The approach was almost level and consisted of two lanes, one for straight ahead and left-turning traffic and the second for straight ahead only with a prohibited right-turn control.

The outward traffic from the city centre was considered for the study purposes during afternoon peak hours.

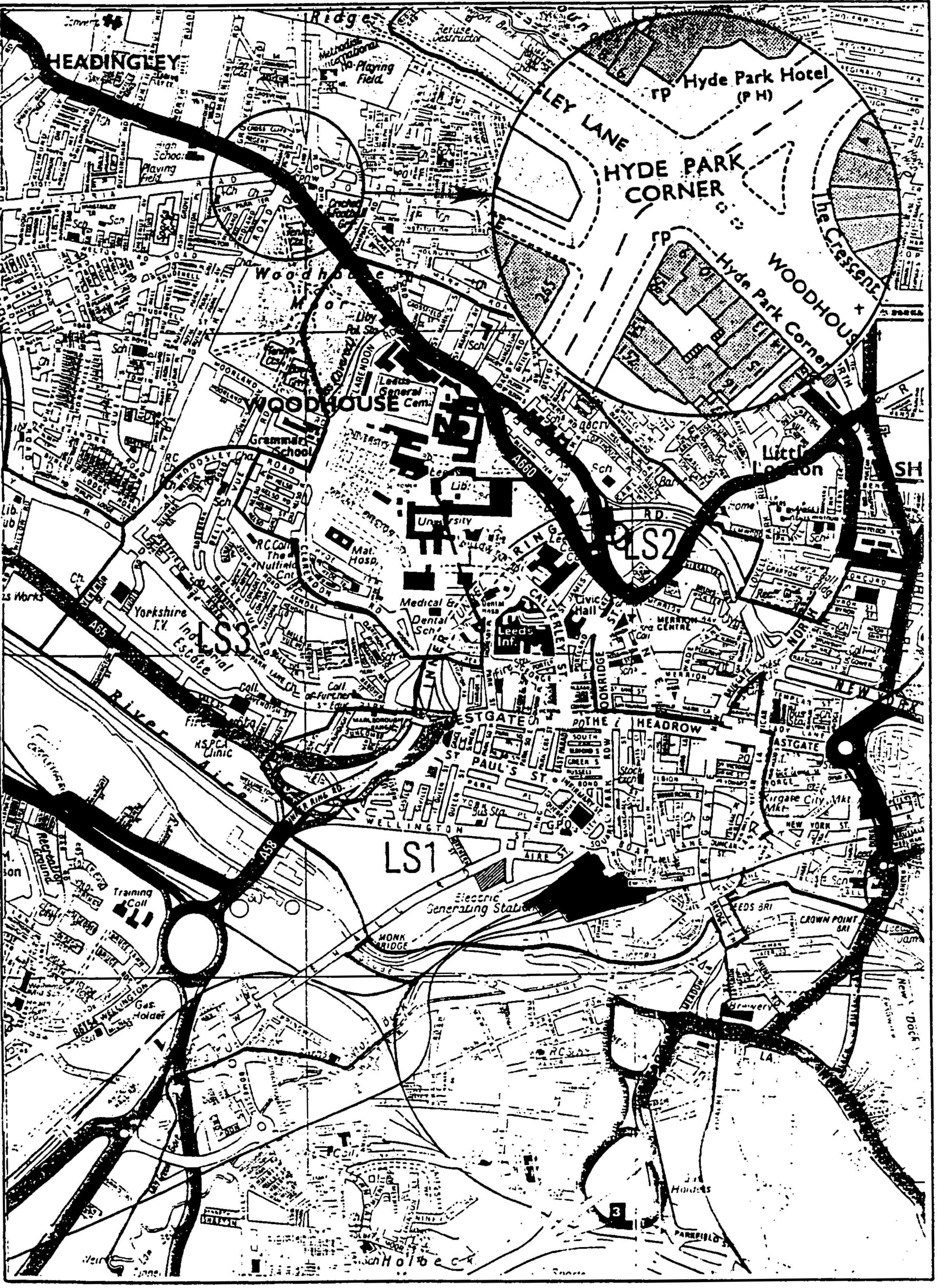


Figure 2.5.3.3 - Woodhouse Lane and Headingley Lane signalized intersection (Leeds)

(4) Headingley Lane (A660)

Traffic Signal Approach, Leeds.

The approach is on the opposite side of Woodhouse Lane approach mentioned in site 3, which is on a main route leading to Leeds City Centre (Fig. 2.5.3-3). The approach was almost level and consisted of two lanes, one for straight ahead and left-turning traffic and the second for right-turning only.

The inward traffic to the city centre was considered for the study purposes during the morning peak hours.

(5) Streatham Hill (A23)

Traffic Signal Approach, London.

Filming was carried out of the north bound carriageway of the approach which was almost level and consisted of three lanes (Fig. 2.5.3-4), which carried traffic from the south of London (Croydon and Surrey) to the north towards Central London. The centre lane was solely for straight ahead traffic, while the off-side lane was for right-turning vehicles only and the near-side lane was for straight ahead and left-turning vehicles. The road was a part of the A23 connecting Brighton in the south with Central London and therefore the road carried an appreciable amount of heavy commercial vehicles and buses passing through the intersection. The intersection as a whole represented an ideal layout for the observation due to its location and its level of saturation during the morning and afternoon peak hours and due to the high percentage of heavy vehicles which was approximately 28%.

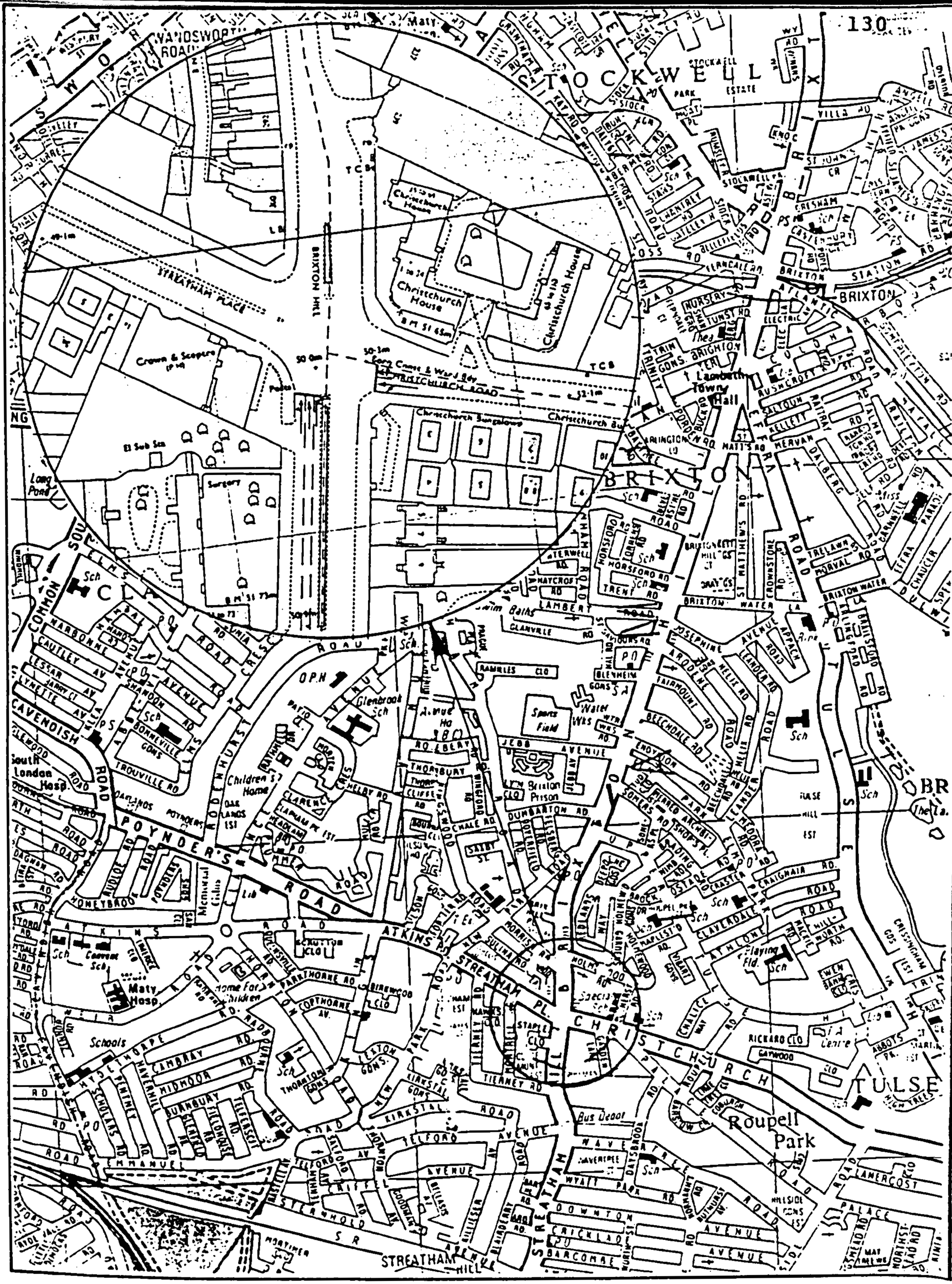


Figure 2.5.3.4 - Streatham Hill and Christchurch road traffic signal approach (London)

(6) Christchurch Road (A205)

Traffic Signal Approach, London.

The Christchurch approach consisted of two lanes, one for straight ahead and left-turning traffic and the second for straight ahead and right-turning vehicles, (Fig. 2.5.3-4). The right-turning vehicles had the advantage of the very wide area of the intersection while negotiating their turning manoeuvre, where about five to six vehicles can wait before turning without affecting the movement of straight ahead vehicles. The approach had a very slight down hill gradient towards the stop line.

The flow of traffic consisted of a large proportion of heavy commercial vehicles, 22% approximately and the approach was fully saturated during morning peak hours.

2.6 - Method of observation

2.6.1 - Introduction

Data necessary for the calculation and analysis of traffic parameters at a signalized type of control may be collected by a variety of methods ranging from simple grouped manual counts to fully automatic recording of events. Generally the more sophisticated the data collection process, the more detailed information and results which can be obtained.

At signalized intersections the information required is based on the timing of saturated green periods and rate of vehicles discharged. This will require precise monitoring equipment to be mounted opposite to the approach under consideration. The standard of accuracy and consistency that were associated with the collection of data had to be high in order to reduce the error rate.

In the laboratory the analysis should be of a high level of accuracy. This was achieved by using advanced equipment to monitor the movement of vehicles.

With regard to accuracy it is important to note that although there is plenty of scope for improvement in the error rate in the primary count data, variability due to sampling in time or space, weather conditions and local traffic events (parked vehicles, accidents, road works, temporary traffic management measure, etc.), will inevitably be present whatever measurement technique is used.

2.6.2 - Filming technique and data collection.

Recording numerous traffic events occuring at the same time i.e. time headway, speed, vehicle counting and its classification, traffic density, ...etc. requires permanent records and special techniques to enable researchers to abstract all the necessary information.

Video images are a very good source of information for the analysis of traffic flows. Detailed studies of vehicle speeds or origin-destination patterns at junctions usually entail labour-intensive manual or complicated semi-automatic methods. The use of video recordings as a way of gathering a permanent record from which data can be abstracted at some later convenient time is becoming popular. Information can be gathered (depending on site location and weather conditions) using the following equipment; video camera, portable video recorder with tapes, tripod and power supply.

Video filming as a source of traffic data collection, is widely used by many researchers and traffic study groups and organisations. Voorheas (60) suggested the use of video filming as explained in (Fig. 2.6.2-2) to obtain all necessary detail about discharge headways of vehicle type, and to distinguish the vehicle types by their physical size and their performance on an urban network.

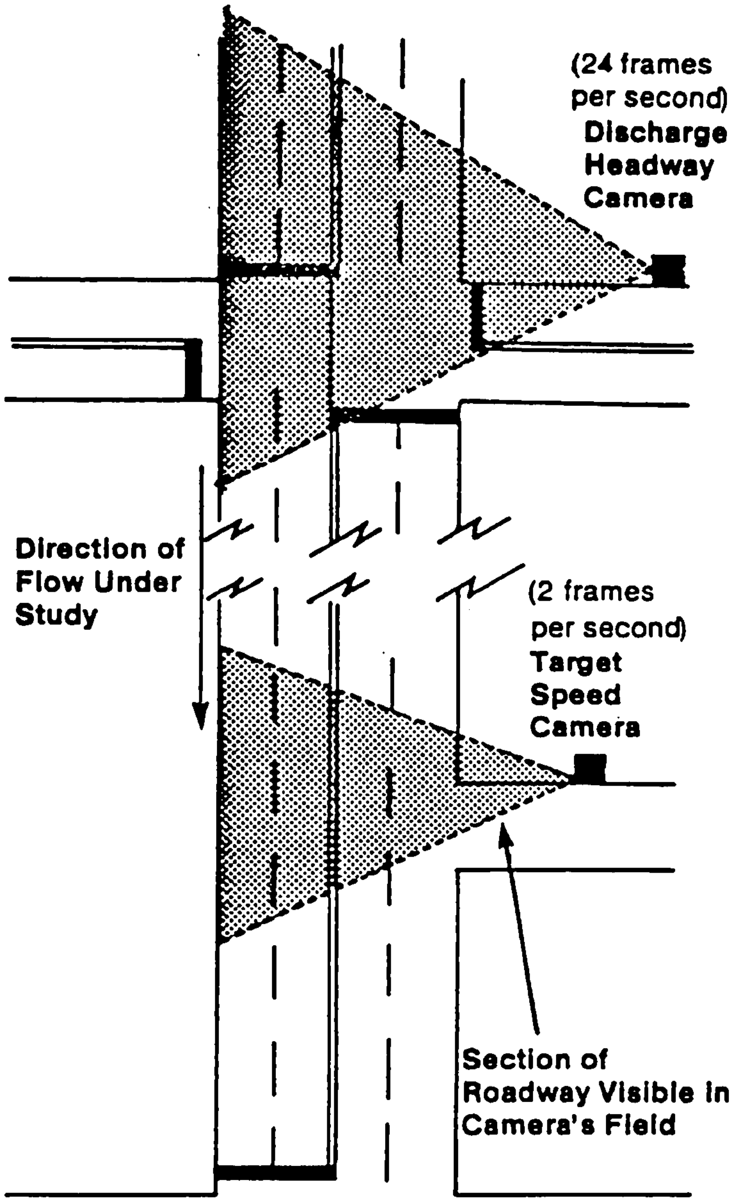


Figure 2.6.2.1 - Typical Video Camera Placement

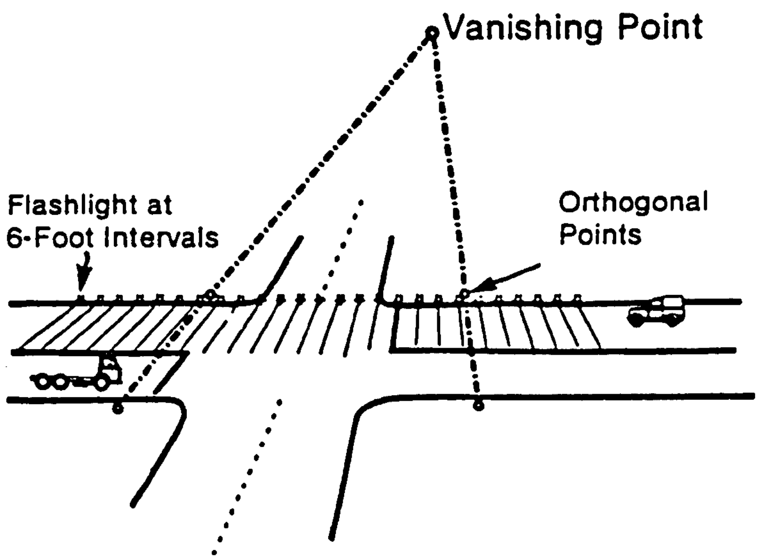


Figure 2.6.2.2 - Distance Measuring Technique

The discharge headway camera was used to collect both discharge headway and acceleration data. The second camera was used to measure target speed and vehicle length. Both cameras were operated at 2 frames per second.

The Australian Road Research Board (68), undertook and sponsored a research project in their current efforts towards studying the movement of vehicles on urban sections of roads and different types of intersections. In addition, to study the capacity and performance of roundabouts the Australian Road Research Board used video filming techniques and developed an analysis procedure of video images using microcomputers. A technique similar to this has been used successfully in the past and has been described by Dods (69).

Recent developments on road traffic surveillance and control systems using digital computers require a large quantity of traffic information to be gathered on a real-time basis. The necessary information is focused on special traffic parameters i.e. the length of queue, the grade of congestion and information on unusual traffic incidents. Real-time processing of moving pictures on traffic flow is considered to be useful for obtaining various types of spacial traffic parameters. It reduces the time and work required in traffic investigation. Hilbert (70), Houkes (71) and Takaba et al (72) have developed a traffic flow measuring system using a video camera. A group at UMIST and the University of Sheffield have developed a traffic data collection system by using a CCD Camera (73). Also in Japan, Ooyama et al (74) and Kudo et al (75) have introduced a system using a sensor of specially arranged photo diode array or the one using a CCD sensor. Field testing with these systems has since been tried.

2.6.3 - The method adopted for data collection.

Video filming technique was used in this study to collect traffic data from various signalized intersections in West Yorkshire and London. A modern colour video camera and portable video recorder were used; they are described in detail in Appendix A.

Because of the sophisticated facility available within the equipment, they were considered to offer a practical and reasonably accurate choice both for filming the various sites and for the process of film analysis in the laboratory.

Several video films (VHS) were used at each site and some difficulties were experienced during filming, such as poor weather conditions which prevented long periods of filming at times.

At Site 1 the observation was carried out for a total of 15 hours during the morning peak periods between 7.30 a.m. and 9.00 a.m. and the camera was located on the opposite side of the approach.

The traffic movements are shown in Figure 2.6.3-1 which represent the common traffic arrangement for a 4-arm

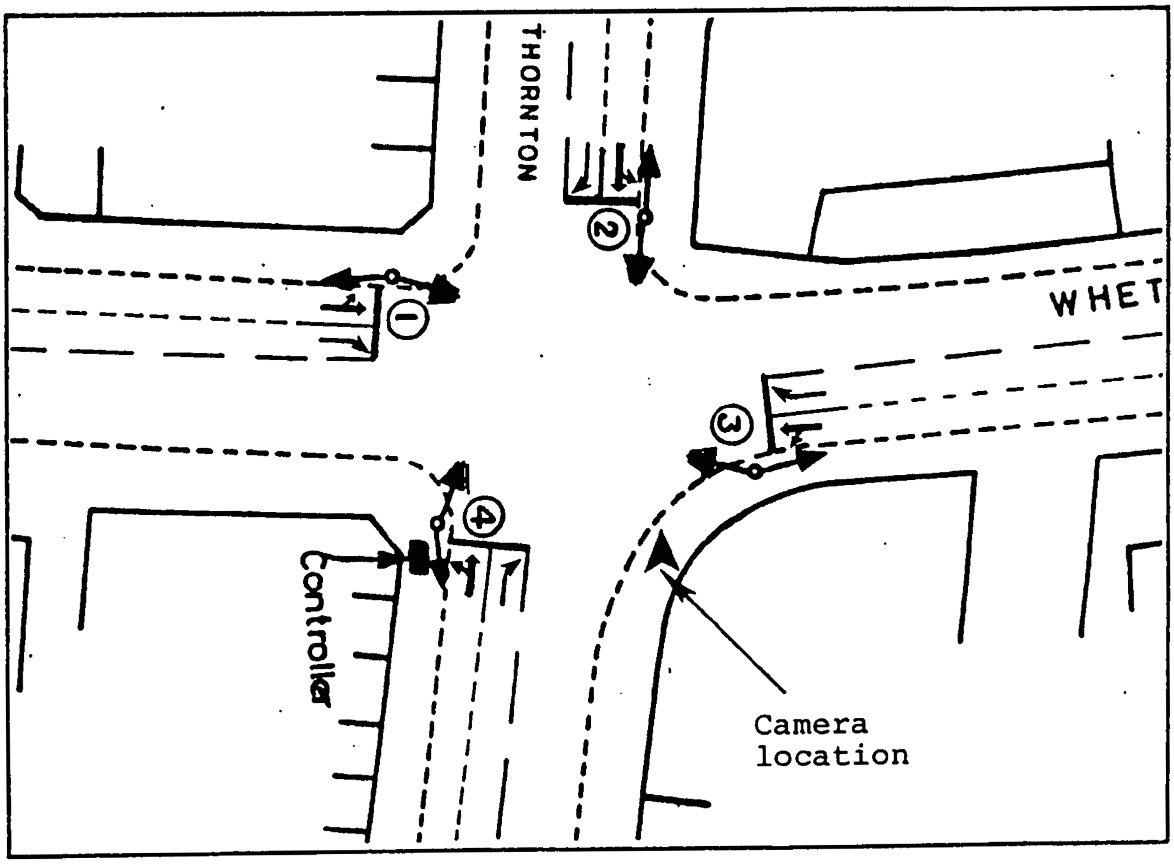


Figure 2.6.3-1 Site "1" junction layouts, camera location and traffic movements (Bradford).

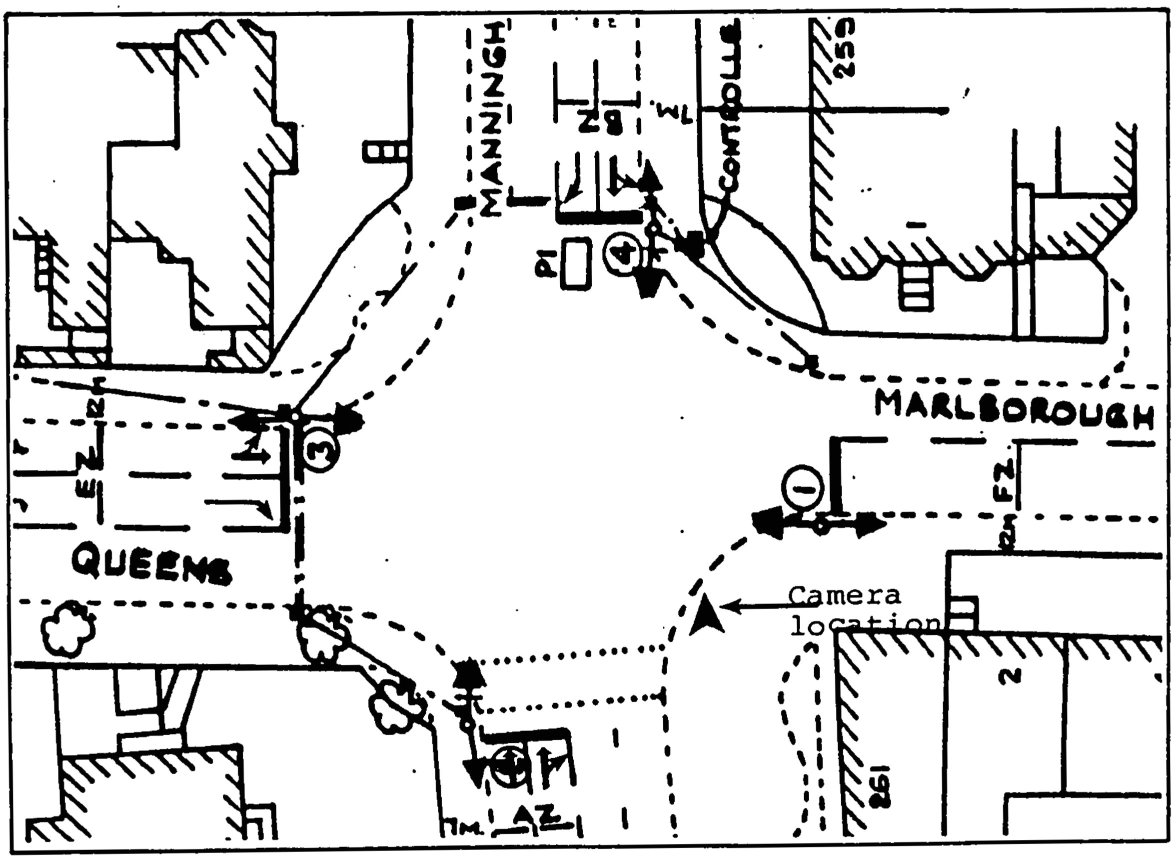


Figure 2.6.3-2 Site "2" junction layouts, camera location and traffic movements (Bradford).

intersection and there was no obstruction caused by the right-turners to straight ahead movement.

The site is free from any physical obstructions and bus stop locations were reasonably away from the immediate vicinity of the intersection.

At Site 2 the observation was carried out for 15 hours collectively during the afternoon peak periods between 3.00 p.m. and 5.00 p.m.

The camera was located on the opposite side of the approach where there is ample space without causing any onstruction to pedestrians, Figure 2.6.3-2.

The observations were taken at times when the approach had reached saturation conditions for traffic leaving the city centre.

Right turning manoeuvre was not causing any obstruction to straight ahead movements and the site was free from any physical obstructions. The bus stop at the approach was located at a reasonable distance from the stop line.

At Site 3 the observation was carried out

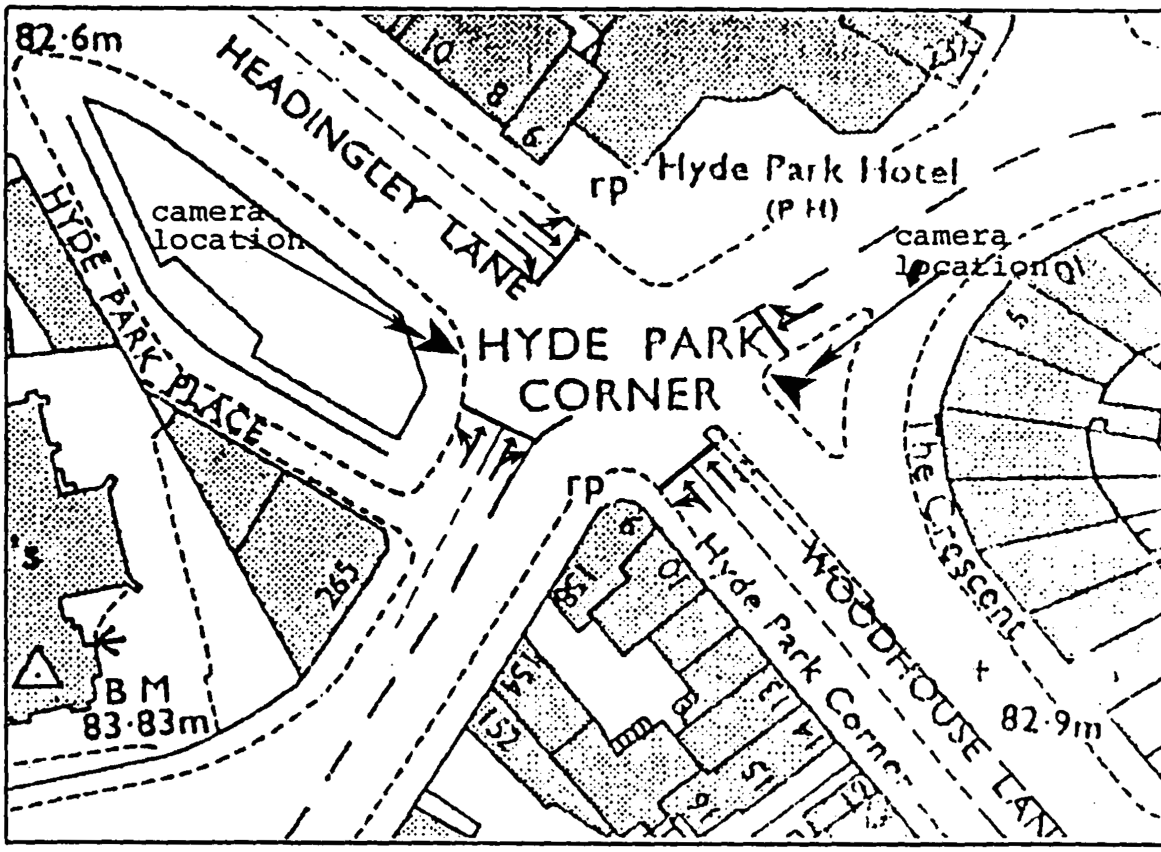


Figure 2.6.3-3 Site 3 and Site 4 junction layouts, camera location and traffic movements (Leeds).

during the morning peak period between 7.30 a.m. and 9.30 a.m. and for 15 hours in all. The camera was located on the opposite side of the approach as shown in Figure 2.6.3-3. The Figure also shows the traffic movements which represent the common arrangement for a 4-arm intersection, except the prohibited right turning movement from the approach under consideration.

The site is free from any physical obstruction and very well marked with clear traffic signs.

Site 4 is the opposite side of site 3 where observation was carried out for 10 hours in total during the morning peak periods between 7.30 a.m. and 9.00 a.m. The camera was located on the opposite side of the approach as shown in Figure 2.6.3-3. Traffic movements are the same as in site 3 except for the off-side lane where it was restricted for right turning vehicles.

At Site 5 the observations were carried out for 20 hours in total during the morning peak period between 7.30 a.m. and 9.30 a.m. The middle lane approach was used by straight ahead traffic only and this was considered in the observation.

The camera was located at the opposite corner of

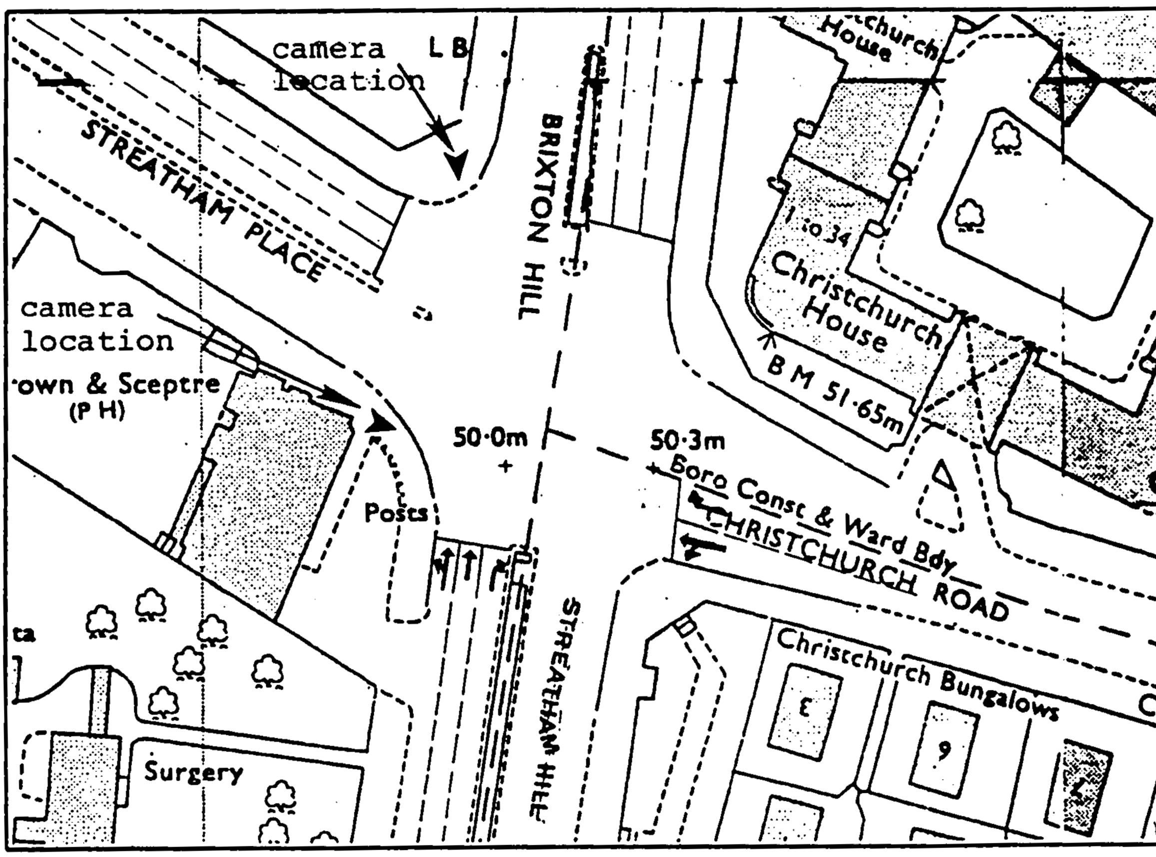


Figure 2.6 .3-4 Site 5 and Site 6 junction layouts, camera location and traffic movements (London).

the approach and due to the width of the intersection it had less effect on driver behaviour as they were leaving the stop line, Figure (2.6.3-4). The observations were carried out without interference from right-turning vehicles.

At Site 6 the observations were carried out for a total of 20 hours during the morning peak period between 7.30 a.m. and 9.00 a.m. The site and traffic movements are as shown in Figure 2.6.3-4, one lane for straight-ahead and left-turning vehicles and the second for straight-ahead and right-turning traffic. The camera was located on the opposite side of the approach where the lane for straight-ahead and left-turning traffic was filmed.

2.6.4 - Film analysis.

The analysis of video films was carried out in the laboratory using the portable colour video cassette recorder (HR-2200 EG/EK JVC) connected to the tuner/adapter (TV-24 EK JVC), with remote control facility and 20 ins. 4 System colour monitor (VM-20PS MCG).

During the analysis process a Hewlett Packard "HP-85" micro-computer was used to store data obtained from recorded video tapes. In order to give a clear view of site and vehicle movements a big screen of 20" monitor was used to display each recorded video tape of 1 hour or 2 hours (VHS). The pictures were advanced frame by frame using the frame advance control facility, see Figure 2.6.4-1, which outlines the equipment used in the laboratory.

An experiment was carried out in the laboratory to measure the number of frames per second. It was found that the normal scan rate was 25 frames per second (fps). This gives an accuracy of 0.04 sec. per frame which allows a high accuracy in measuring time intervals between successive events. To aid the analysis of the large number of films taken, special analysis sheets were prepared on which different events were recorded in tabular form together with the use of a micro-computer to store data.

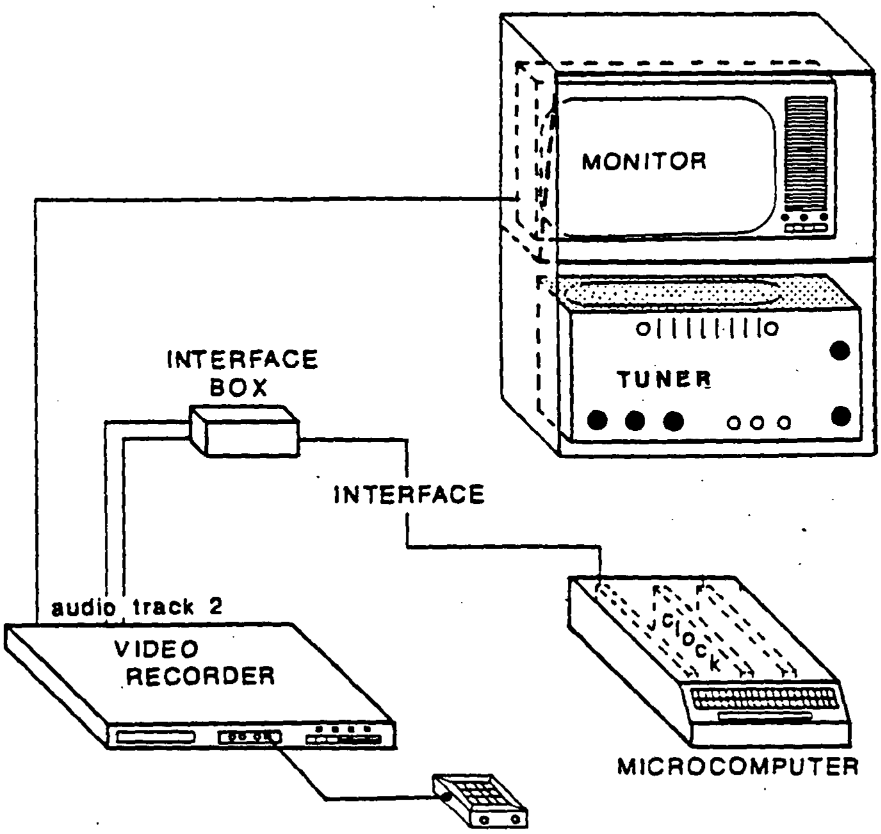


Figure 2.6.4 - 1 Analysis equipment

The events extracted from recorded films were as follows:-

(i) Number of successive vehicles per green time.

(ii) Vehicle type.

(iii) Green time.

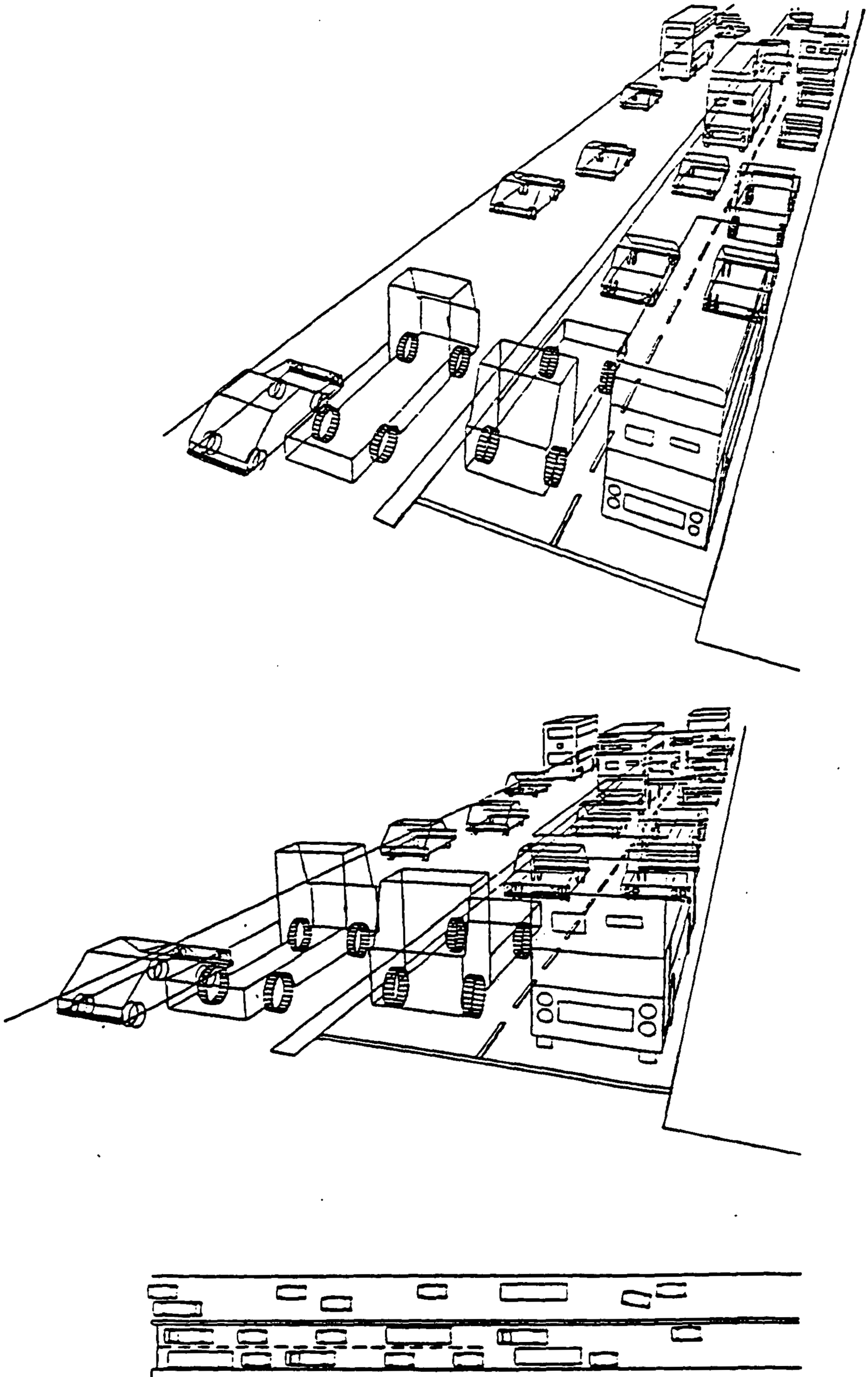
(iv) Start and end lost times.

(v) Number of left turning vehicles and straight ahead only.

(vi) Time headway between successive vehicles.

The type of each vehicle leaving the stop-line was noted in order to investigate the effect of vehicle type on highway traffic flow. Time headway between successive vehicles were recorded as front wheels of each vehicle crossed the stop-line.

Computer images of video films analysis are shown in Figure 2.6.4-2, which gives a comparison between two different angles of filming the same site (a) and (b). The ideal angle of filming is shown in (b).



(a)

(b)

(c)

Figure 2.6.4. - 2 Computer images plan and two isometric views of the same traffic signal approach.

2.7 - Data analysis

2.7.1 - Vehicle-Type Definition.

During the analysis of films, the numbers of various vehicle types are counted in order to calculate PCU values. For this process it is necessary to adopt a specific category of vehicle classes to be able to distinguish between different types of vehicles. Vehicle classification has been reproduced as shown in Figure (2.7.1-1). This classification, which meets the requirements of the Department of Transport, as defined in EEC regulation R1108/70, has been adopted in this study. This classification was produced by the Transport and Road Research Laboratory in collaboration with the Department of Transport and Scottish Development Department. Two hundred and twenty locations were chosen throughout the U.K. as study sites in rural and urban areas ( ). Vehicles were classified into a total of 20 types; they were given code numbers for analysis and automatic vehicle classification purposes. In addition, if an acceptable vehicle parameter match could not be achieved, vehicle were then classified into 6 additional categories by the number of axles.

The classification in this study was connected to TRRL classes shown in Figure (2.7.1-1) by indicating the class number related to it.

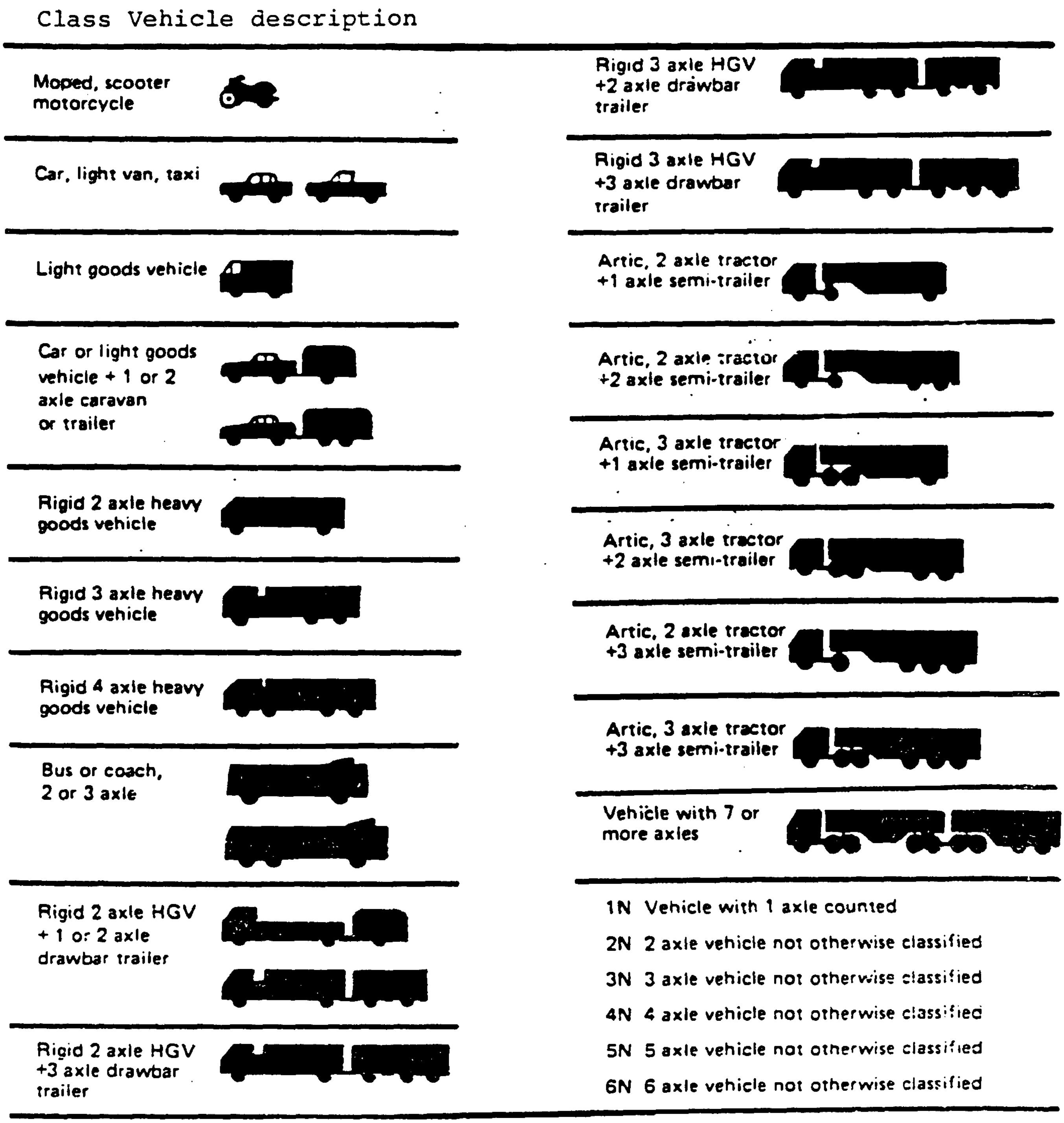


Figure 2.7.1-1 TRRL vehicle class listing compatible with EEC Regulation R1108/70

(Reproduced from reference No.2)

To simplify the analysis in this study vehicle-type classes were grouped into the following categories and are used throughout:-

((a)) "Cars" (C), light vehicles (3 or 4 wheeled vehicles) including light vans with 2-axles. TRRL Class 2N.

((b)) "Light Goods Vehicles" (LGV), including medium and large vans pr medium commercial vehicles with 2-axles but more than 4 wheels. TRRL Class 2N.

((c)) "Heavy Goods Vehicles" (HGV), vehicles with 2-axles or more, long-wheel base lorries, coaches and buses (single and double deck). TRRL Class 3N.

((d)) "Articulated Goods Vehicles" (AGV), vehicles with 3-axles, 4axles or more, including semi-trailer and trailer-coach. TRRL Classes 4N, 5N and 6N.

((e)) "Motorcycle" (MC), moped, scooter with two wheels. TRRL Class 1N.

Also cars and vans turning to the left at junctions were indicated as C_L and LGV_L, to calculate their effect on PCU values.

2.7.2 - Theoretical Basis of the Analysis.

The basis of the analysis is that the saturation flow at a signalized intersection is a constant value and the total number of vehicles flowing through the intersection in unit time is dependent upon the proportions of different types of vehicles in the flow.

This can be expressed simply as:

Saturation Flow = (The number of vehicles of type "n"X. The passenger car unit for vehicles type "n")................. 2.7.2-1

or

$$S.F. = (X_1\ A_1)+(X_2\ A_2)+(X_3\ A_3)\ \ldots\ldots +(X_n\ A_n) \ldots 2.7.2\text{-}2$$

where

S.F. = Saturation flow in pcu's in time t secs.

Xn = The number of vehicles of type n.

An = The passenger car unit for vehicles type n.

By counting the total number of vehicle types passing the stop-line during a saturation flow period, the following equation was introduced to calculate passenger car unit values:

$$SF = a_0+a_1X_1+a_2X_2+a_3X_3+\ \ldots\ldots\ +a_iX_i\ \ldots\ldots\ 2.7.2\text{-}3$$

where

SF = Saturation flow period in seconds,

a_0 = The constant parameter,

$X_1, X_2, X_3 \ldots X_i$ = Vehicle types,

$a_1, a_2, a_3 \ldots a_i$ = Coefficient parameters (p.c.u. weightings for vehicle types $X_1, X_2, X_3 \ldots X_i$).

Multiple linear regression analysis is used to compute PCU weightings for each vehicle type considered and is also used to test the significance of the results. Regression analysis is also used for testing the relationship between a dependent variable (saturation flow period) and two or more independent variables (number of vehicle types).

Since there are five vehicle types considered in the study (passenger cars, light goods vehicles, heavy goods vehicles, buses/articulated goods vehicles and motorcycles), cars turning left and light goods vehicles turning left, there will be seven separate vehicle types or seven independent variables in the above equation. The final equation which was used in the regression analysis is as follows:-

$$SF = a_0 + a_1X_1 + a_3X_3 + a_4X_4 + a_5X_5 + a_6X_6 + a_7X_7 \quad \ldots\ldots \quad 2.7.2\text{-}4$$

where:

SF = Saturation flow period in seconds.

a_o = Constant parameter.

X_1 = Number of passenger cars.

X_2 = Number of light goods vehicles.

X_3 = Number of heavy goods vehicles.

X_4 = Number of buses and articulated goods vehicles.

X_5 = Number of motorcycles.

X_6 = Number of passenger cars turning left.

X_7 = Number of light goods vehicles turning left.

a_1, a_2, a_3 ..... a_7 = Passenger car unit weightings for the above mentioned vehicle types respectively.

These are all known quantities, and a_1, a_2 and a_7 are the unknown PCU factors.

Only passenger cars and light goods vehicles turning left were considered in this study because of their significant numbers, while other types of vehicles turning left were very few in number, so the effect of their values would be insignificant.

2.7.3 - Computer programming and analysis.

The Statistical Package for the Social Sciences (SPSS) program was used for data analysis. The package comprises a powerful set of statistical programs and was written originally for social scientists, but has since been expanded and now includes many general-purpose statistical routines.

SPSS (76) is designed so that the user can perform statistical analysis using regression. The procedure REGRESSION derives the best multiple linear regression of the dependent variable on a set of independent variables. The number of independent variables may be fixed or variables may be introduced one at a time (Stepwise regression) until a satisfactory regression has been obtained.

The three types of multiple regression with variable selection were used in the analysis; they are:-

i - Forward method (Stepwide) inclusion;
ii - Stepwise Multiple Regression;
iii - Backward Elimination.

Figure (2.7.3-1), illustrates the overall flow chart outlines, which show the initializing and the readings of the control cards of the program then entering

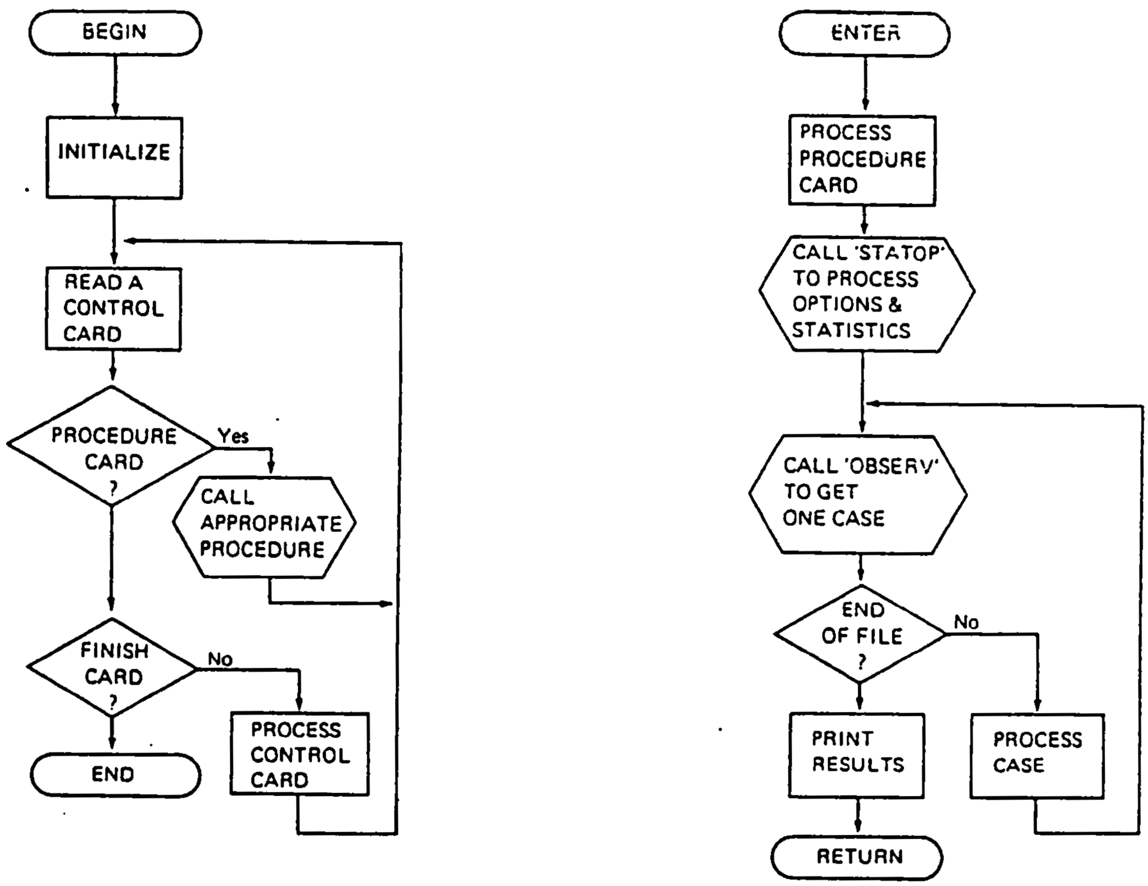


Figure 2.7.3-1 Overall system flow chart

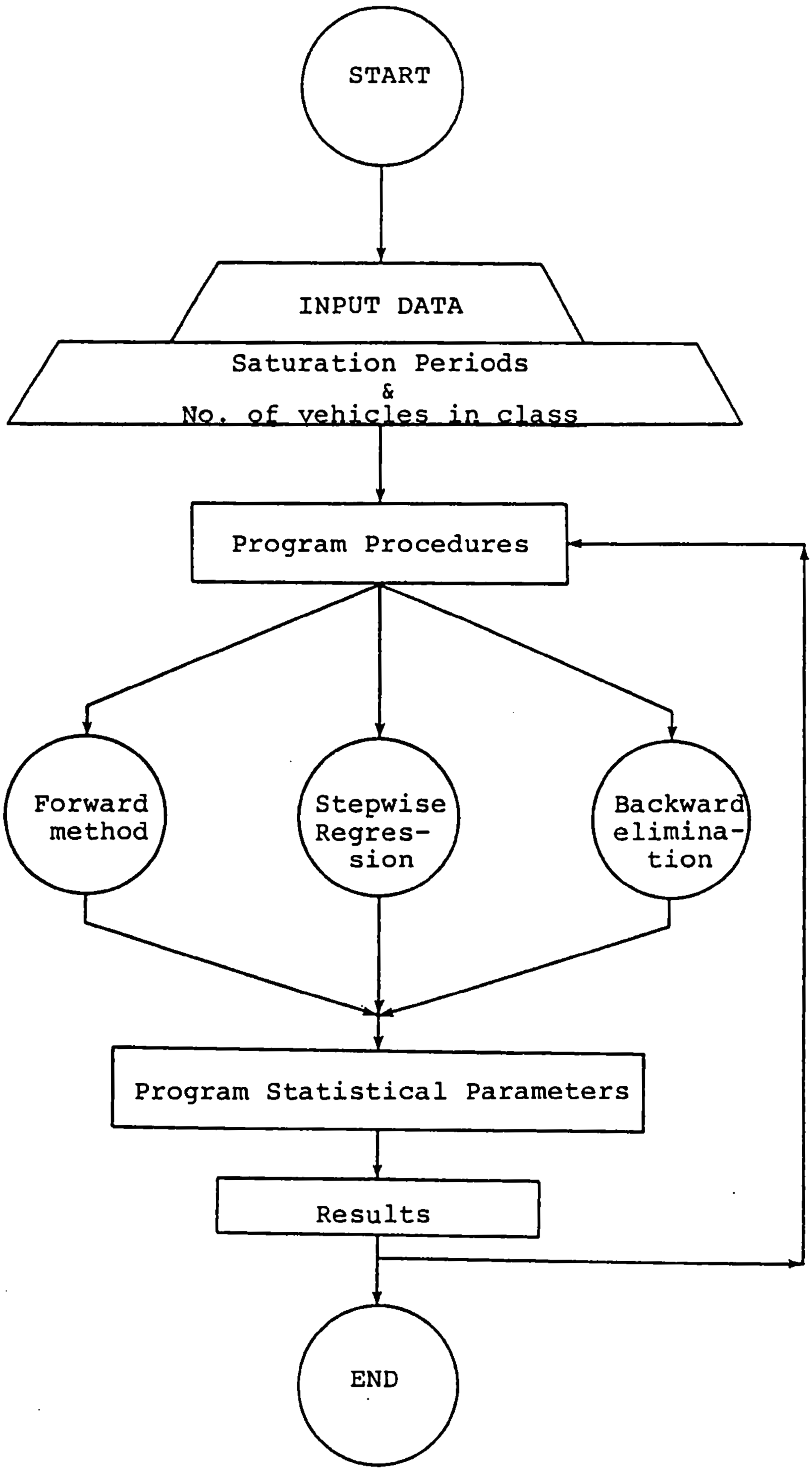


Figure 2.7.3-2 Flow Chart of Main Computing Program

the steps of options and statistics and data file readings (77).

Figure (2.7.3-2), illustrates the outline program procedures for the three methods of regression used to analyse stored data for every site under study.

The program was used to determine the passenger car units for each set of data. Equation (2.7.2-4) was used in which the independent variables were the initial input to the program, in this case the number of each type of vehicle considered at the saturated period. As an example, the first reading for site (1) is:

Saturated green time (seconds)	Car	LGV	HGV & Buses	AGV	MC	C_L	LGV_L	TOTAL
28.12	5	1	1	1	2	1	1	12

Appendix "B" shows more illustrated data file samples.

Traffic data were stored in microcomputer during film analysis and then fed to the CYBER computer at the University of Bradford by direct entry through interpreting the word "card" to mean a line of terminal-input.

The results for the passenger car unit values and the standard errors for each vehicle type are shown in tables (2.7.3-1) to (2.7.3-6). Each table represents the results for each site using the Forward, Backward and Stepwise Regression methods.

Tables (2.7.3-7) to (2.7.3-12) show the mean values for the results.

2.8 - Presentation of results

2.8.1 - Introduction.

Results were obtained utilizing the analytical methods outlined in the previous paragraph (2.7.3), and are presented in both a tabular and a diagrammatic form where appropriate. Results from the computer out-put are given in Appendix B, showing all statistical values related to the calculation of PCU factors. Table (2.8.1-1) shows a summary of site study and traffic movements under investigation at each site. Table (2.8.1-2) gives a summary of one hour film analysis of each site showing total number of vehicles and number of each vehicle type with their percentages and the total time in seconds of observation together with the average times per observation.

SITES SURVEYED

Site No.	Location	Site[1]	Direction[2]	Turning Movements Investigated.[3]
1	BRADFORD	Thornton Road/Ingleby Road	E	AL
2	BRADFORD	Manningham Lane/Queens Road	N	AL
3	LEEDS	Woodhouse Lane/Hyde Park Road	N	AL
4	LEEDS	Headingley Lane/Woodhouse Street	S	AL
5	LONDON	Streatham Hill/Streatham Place	N	A
6	LONDON	Christchurch/Brixton Hill	W	AL

1 The first road listed under 'SITE' is the one investigated.

2 N = Northbound; E = Eastbound; S = Southbound;
W = Westbound (for ahead-only movements.)

3 A = Ahead only; AL = Ahead/Left.

Table 2.8.1-1 Summary of the study sites

SITE	Condition at Filming	Cars	Light Goods Vehicle	Heavy Goods Vehicle	Artic. Goods Vehicle	Motor Cycle	Cars (turning left)	Light Goods Vehicle (turning left)	Total Vehicles	Total time (secs.)	No. of Obs.	Average Time per Obs. (secs.)
(1)	Dry and Clear	695 53%	122 9%	118 9%	118 9%	74 6%	94 7%	85 7%	1306	1593.38	60	26.56
(2)	Dry and Clear	848 55%	147 10%	130 8%	129 8%	86 6%	103 7%	97 6%	1540	1778.47	67	26.54
(3)	Dry and Clear	1028 63%	114 7%	123 7%	138 9%	78 5%	84 5%	68 4%	1633	1838.74	63	29.17

Table (2.8.1-2)

Summary of one hour sample data showing totals and percentages of vehicles in the various classes together with average times per observation.

Continued.

SITE	Condition at Filming	Cars	Light Goods Vehicle	Heavy Goods Vehicle	Artic. Goods Vehicle	Motor Cycle	Cars (turning left)	Light Goods Vehicle (turning left)	Total Vehicles	Total time (secs.)	No. of Obs.	Average Time per Obs. (secs.)
(4)	Dry and Clear	1128 66%	130 7%	130 7%	150 9%	89 5%	94 6%	- -	1721	1968.12	69	28.52
(5)	Dry and Clear	1238 68%	176 10%	136 7%	122 7%	147 8%	- -	- -	1819	1961.75	74	26.51
(6)	Dry and Clear	898 56%	113 7%	113 7%	129 8%	93 6%	139 8%	130 8%	1615	1847.58	76	24.31

Table 2.8.1-2 Continued.

Summary of one hour sample data showing totals and percentages of vehicles in the various classes together with average times per observation.

2.8.2 - Saturation flows and lost time

Saturation flows were obtained from the analysis of the observed data at the six sites and they are shown in table 2.8.2-1. Description of the sites and method of observation are discussed in sections 2.5 and 2.6 of this chapter. At the approaches there were continuous queues even at the end of the green period. The composition of the traffic at each site is as shown in table 2.8.1-2.

It was found that saturation flows cary between 1,760 to 1,996 pcu/h. This variation is due to the approaches' geometrical features and traffic composition.

Saturation flows were found to be constant throughout the green period at all sites. This is because the traffic queue formation was confined to the approach lane.

Figure 2.8.2-1 shows the lost time distribution between sites. The average starting lost time was 1.356s and the standard deviation over the site-mean values was 0.535s; corresponding figures for the end lost time were 0.837s and 1.018s respectively. These values are lower than the accepted value of 2s which indicates higher manoeuvring performance by all vehicle types in the present composition of the traffic flow.

SITE	Approach Width (m)	Number of Lanes	Average Green Time (secs.)	Observed Av. Initial Lost Time (secs.)	Observed Av. End Lost Time (secs.)	Number of Observations	Effective Green Time (secs.)	Saturation Flow (PCU/HR/LN)
Thornton Rd./ Whetlet Lane (Bradford)	3.66	2	28	1.55	0.13	219	26.32	1880
Manningham Lane (Bradford)	4.00	2	32	1.75	0.22	176	30.03	1976
Woodhouse Lane (Leeds)	3.13	2	34	2.22	0.15	220	31.63	1773
Headingley Lane (Leeds)	3.06	2	34	1.97	0.28	154	31.75	1960
Streatham Hill (London)	3.05	3	38	1.67	0.45	119	35.88	1996
Christchurch (London)	3.81	2	29	2.18	0.57	143	26.25	1947

Table 2.8.2-1 Analysis of traffic parameters showing saturation flows, green time and lost times.

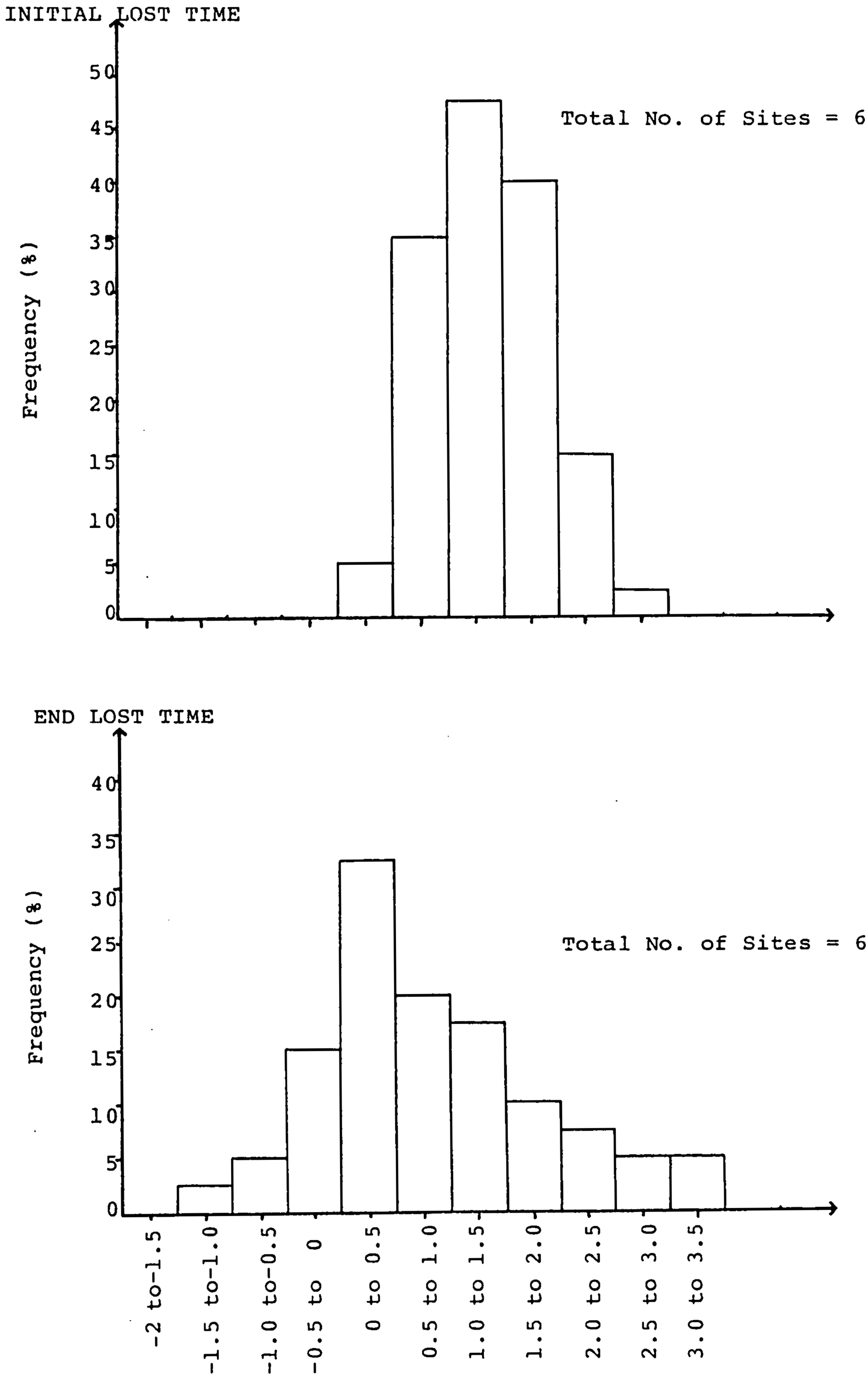


Figure 2.8.2-1 Initial and End Lost Times Distribution

2.8.3 - Passenger car unit values

The analytical method outlined in section 2.7.2 of this chapter was used to obtain passenger car unit values by utilizing the data obtained from six signalized intersections in Bradford, Leeds and London. Equation 2.7.2-4 was used in a multiple regression computer program (SPSS) to compute the coefficients for each variable. For each signalized intersection a set of 100 equations were used as an input to the computer program. The number of independent variables (number of vehicles in each class) in the regression equation vary from site to site according to the types of movement considered. For sites (1), (2), (3) and (6) the number of independent variables considered were seven, these include passenger cars, light goods vehicles, heavy goods vehicles, buses/articulated goods vehicles, motor cycles, cars and light goods vehicles turning left. At site (4) the number of independent variables considered were six. They are the same as those in the previous sites except for light goods vehicles turning left. Because the lane considered at the approach of site (5) was exclusively for straight-ahead movements the number of independent variables were five. These comprise passenger cars, light goods vehicles, heavy goods vehicles, buses/ articulated goods vehicles and motor cyles, see table (2.8.1-1). Samples of computer inputs are given in

Appendix B and samples of computer outputs are given in Appendix C .

The three types of multiple regression analysis used to calculate passenger car unit values and their statistical results are as shown in the computer output in Appendix C . These results are summarized and presented in tabular form. For each site investigated the results are given in two tables. The first shows the results obtained from the three regression methods outlined in section 2.7.3 of this chapter and the second table gives values of the results.

Table 2.8.3-1 shows the results of computed passenger car unit values for site (1) (Thornton Road, Bradford). Table 2.8.3-2 shows the average values of passenger car units. Using average PCU values the final equation for site (1) is as follows:

$$S.F. = X_1 \quad +1.11X_2+1.70X_3+2.10X_4+0.35X_5+1.12X_6+1.20X_7$$

$$F = (1052)\ (1601)\ (2661)\ (3545)\ (61.3)\ (1029)\ (765.3)$$

$$t = (102)\ (40.0)\ (51.6)\ (59.5)\ (7.83)\ (27.7)\ (27.7)$$

SITE (1)

Vehicle Type	Passenger Car Unit Values (standard errors are given in brackets)		
	Forward Regression	Backward Regression	Stepwise Regression
Passenger Car C	1.00 (0.0097)	1.00 (0.016)	1.00 (0.016)
Light Goods Vehicle LGV	1.12 (0.028)	1.11 (0.036)	1.11 (0.035)
Heavy Goods Vehicle HGV	1.70 (0.033)	1.70 (0.042)	1.70 (0.042)
Articulated Goods Vehicle & Buses AGV & B	2.10 (0.034)	2.10 (0.039)	2.10 (0.039)
Motor Cycle MC	0.36 (0.045)	0.34 (0.047)	0.34 (0.048)
Passenger Car (turning left) C_L	1.12 (0.035)	1.12 (0.037)	1.12 (0.037)
Light Goods Vehicle (turning left) LGV_L	1.22 (0.044)	1.20 (0.051)	1.20 (0.051)

Table 2.8.3-1 Passenger car unit values at Thornton Road, Bradford.

SITE (1)

Vehicle Type	Mean PCU's Values (standard errors are in brackets)
PC	1.00 (0.0139)
LGV	1.113 (0.033)
HGV	1.70 (0.039)
AGV	2.10 (0.0373)
MC	0.347 (0.047)
CL	1.12 (0.036)
LGV_L	1.207 (0.049)

Table (2.8.3-2) Mean Passenger Car Unit Values from computer regression analysis

SITE (2)

Vehicle Type	Passenger Car Unit Values (standard errors given in brackets)		
	Forward Regression	Backward Regression	Stepwise Regression
Passenger Car C	1.00 (0.015)	1.00 (0.022)	1.00 (0.022)
Light Goods Vehicle LGV	1.09 (0.046)	1.10 (0.050)	1.10 (0.050)
Heavy Goods Vehicle HGV	1.81 (0.050)	1.78 (0.045)	1.78 (0.050)
Articulated Goods Vehicle & Buses AGV & B	1.90 (0.057)	1.90 (0.050)	1.90 (0.064)
Motor Cycle MC	0.55 (0.071)	0.56 (0.072)	0.56 (0.072)
Passenger Car (turning left) C_L	1.12 (0.056)	1.12 (0.056)	1.12 (0.057)
Light Goods Vehicle (turning left) LGV_L	1.20 (0.078)	1.20 (0.079)	1.20 (0.079)

Table 2.8.3-3 Passenger car unit values at Manningham Lane, Bradford.

SITE (2)

Vehicle Type	Mean PCU's Values (standard errors are in brackets)
PC	1.00 (0.0197)
LGV	1.097 (0.049)
HGV	1.79 (0.048)
AGV	1.90 (0.057)
MC	0.56 (0.072)
CL	1.12 (0.056)
LGV_L	1.20 (0.079)

Table (2.8.3-4) Mean Passenger Car Unit Values from computer regression analysis

SITE (3)

Vehicle Type	Passenger Car Unit Values (standard errors are given in brackets)		
	Forward Regression	Backward Regression	Stepwise Regression
Passenger Car C	1.00 (0.009)	1.00 (0.030)	1.00 (0.030)
Light Goods Vehicle LGV	1.05 (0.032)	1.05 (0.047)	1.05 (0.047)
Heavy Goods Vehicle HGV	1.75 (0.036)	1.76 (0.049)	1.76 (0.049)
Articulated Goods Vehicle & Buses AGV & B	2.01 (0.034)	2.01 (0.047)	2.01 (0.047)
Motor Cycle MC	0.31 (0.053)	0.30 (0.055)	0.30 (0.055)
Passenger Car (turning left) C_L	1.02 (0.051)	1.02 (0.054)	1.02 (0.054)
Light Goods Vehicle (turning left) LGV_L	1.20 (0.064)	1.17 (0.071)	1.17 (0.071)

Table 2.8.3-5 Passenger car unit values at Woodhouse Lane, Leeds.

SITE (3)

Vehicle Type	Mean PCU's Values (standard errors are in brackets)
PC	1.00 (0.023)
LGV	1.05 (0.042)
HGV	1.76 (0.045)
AGV	2.01 (0.043)
MC	0. 303 (0.054)
CL	1.02 (0.053)
LG V_L	1.18 (0.069)

Table (2.8.3-6) Mean Passenger Car Unit Values from computer regression analysis

SITE (4)

Vehicle Type	Passenger Car Unit Values (standard errors are given in brackets)		
	Forward Regression	Backward Regression	Stepwise Regression
Passenger Car C	1.00 (0.0078)	1.00 (0.026)	1.00 (0.026)
Light Goods Vehicle LGV	1.16 (0.029)	1.16 (0.039)	1.16 (0.039)
Heavy Goods Vehicle HGV	1.71 (0.031)	1.73 (0.042)	1.73 (0.042)
Articulated Goods Vehicle & Buses AGV & B	2.04 (0.029)	2.09 (0.042)	2.09 (0.042)
Motor Cycle MC	0.42 (0.047)	0.41 (0.047)	0.41 (0.047)
Passenger Car (turning left) C_L	1.11 (0.045)	1.14 (0.046)	1.14 (0.045)
Light Goods Vehicle (turning left) LGV_L	-- --	-- --	-- --

Table 2.8.3-7 Passenger car unit values at Headingley Lane, Leeds.

SITE (4)

Vehicle Type	Mean PCU's Values (standard errors are in brackets)
PC	1.00 (0.020)
LGV	1.16 (0.037)
HGV	1.72 (0.038)
AGV	2.07 (0.038)
MC	0.41 (0.047)
CL	1.13 (0.045)
LGV_L	-- --

Table (2.8.3-8) Mean Passenger Car Unit Values from computer regression analysis

SITE (5)

Vehicle Type	Passenger Car Unit Values (standard errors are given in brackets)		
	Forward Regression	Backward Regression	Stepwise Regression
Passenger Car C	1.00 (0.011)	1.00 (0.039)	1.00 (0.039)
Light Goods Vehicle LGV	1.00 (0.053)	1.00 (0.067)	1.00 (0.067)
Heavy Goods Vehicle HGV	1.60 (0.066)	1.64 (0.074)	1.64 (0.074)
Articulated Goods Vehicle & Buses AGV & B	1.94 (0.069)	1.99 (0.075)	1.99 (0.075)
Motor Cycle MC	0.30 (0.059)	0.30 (0.059)	0.30 (0.059)
Passenger Car (turning left) C_L	-- --	-- --	-- --
Light Goods Vehicle (turning left) LGV_L	-- --	-- --	-- --

Table 2.8.3-9 Passenger car unit values at Streatham Hill, London.

SITE (5)

Vehicle Type	Mean PCU's Values (standard errors are in brackets)
PC	1.00 (0.03)
LGV	1.00 (0.062)
HGV	1.63 (0.071)
AGV	1.97 (0.073)
MC	0.30 (0.059)
CL	-- --
LGV_L	-- --

Table (2.8.3-10) Mean Passenger Car Unit Values from computer regression analysis

SITE (6)

Vehicle Type	Passenger Car Unit Values (standard errors are given in brackets)		
	Forward Regression	Backward Regression	Stepwise Regression
Passenger Car C	1.00 (0.0089)	1.00 (0.028)	1.00 (0.028)
Light Goods Vehicle LGV	1.18 (0.035)	1.18 (0.058)	1.18 (0.058)
Heavy Goods Vehicle HGV	1.68 (0.031)	1.67 (0.039)	1.67 (0.039)
Articulated Goods Vehicle & Buses AGV & B	2.00 (0.034)	1.98 (0.039)	1.98 (0.039)
Motor Cycle MC	0.32 (0.046)	0.32 (0.048)	0.32 (0.048)
Passenger Car (turning left) C_L	1.03 (0.036)	1.03 (0.040)	1.03 (0.040)
Light Goods Vehicle (turning left) LGV_L	1.22 (0.034)	1.23 (0.038)	1.23 (0.038)

Table 2.8.3-11 Passenger car unit values at Christchurch, London.

SITE (6)

Vehicle Type	Mean PCU's Values (standard errors are in brackets)
PC	1.00 (0.022)
LGV	1.18 (0.05)
HGV	1.67 (0.036)
AGV	1.99 (0.037)
MC	0.32 (0.047)
CL	1.03 (0.039)
LGV_L	1.23 (0.037)

Table (2.8.3-12) Mean Passenger Car Unit Values from computer regression analysis

where X_1, X_2, X_3 ... X_7 are number of vehicle types considered at each site.

The "F" and "t" values given in the brackets were found to be highly significant at levels of 0.01 and 0.001 per cent. They exceeded the tabular values of "F" which equals 2.96 and 4.04 respectively and "t" which equals 3.143 and 5.208 respectively.

To test the overall significance of the regression, the following values were obtained:

$R = 0.99993$ $R^2 = 0.99986$ Standard error = 0.3085

$F = 99016.86$ Significant $F = 0.00$

The "F" value exceeded the tabular value at levels of 0.01 and 0.001 per cent so it is highly significant. Therefore the hypothesis is accepted that the regression parameters are significantly related.

Similarly the final equations for the remainder of the five sites and their statistical results are as follows:

i - Site (2), (Manningham Lane, Bradford).

$$S.F. = X_1 \quad +1.80X_2+1.79X_3+1.9X_4 \ +0.56X_5+1.12X_6+1.20X_7$$

$$F \ = (4053) \quad (550) \ (1301) \ (1086) \ (61.6) \quad (411) \quad (231)$$

$$t \ = (63.7) \ (23.5) \ (36.1) \ (32.9) \ (7.85) \ (20.3) \ (15.2)$$

$R = 0.99988$ $R^2 = 0.99977$ STD = 0.44294

F (overall) = 57574 Significant F = 0.00

F (tabular) at 0.01 per cent = 2.96

F (tabular) at 0.001 per cent = 4.04

t (tabular) at 0.01 per cent = 3.143

t (tabular) at 0.001 per cent = 5.208

ii - Site (3), (Woodhouse Lane, Leeds).

$$S.F. = X_1 \quad + 1.05X_2+1.76X_3+2.01X_4+0.30X_5+1.02X_6 +1.18X_7$$

$$F \ = (12377) \quad (1063) \ (2412) \ (3453) \ (33.9) \ (397.8) \ (341.2)$$

$$t \ = (111.2) \quad (32.6) \ (49.1) \ (58.8) \ (5.83) \quad (19.9) \quad (18.5)$$

$R = 0.99994$ $R^2 = 0.99988$ STD = 0.33083

F (overall) = 113616.73 Significant F = 0.000

F (tabular) at 0.01 and at 0.001 per cent = 2.96 and 4.04 respectively

t (tabular) at 0.01 and at 0.001 per cent = 3.143 and 5.208 respectively.

iii - Site (4), (Headingley Lane, Leeds).

$$S.F. = X_1 + 1.16X_2 + 1.72X_3 + 2.07X_4 + 0.41X_5 + 1.13X_6$$

F = (15977) (1545) (2911) (4562) (77.6) (578)

t = (126.4) (39.3) (53.9) (67.5) (8.81) (24.1)

R = 0.99995 R^2 = 0.99990 STD = 0.29297

F (overall) = 156252.06 Significant F = 0.000

F (tabular) at 0.01 and at 0.001 per cent = as above

t (tabular) at 0.01 and at 0.001 per cent = as above

iv - Site (5), (Streatham Hill, London).

$$S.F. = X_1 + X_2 + 1.63X_3 + 1.97X_4 + 0.30X_5$$

F = (9301.95) (368.17) (607.31) (786.57) (25.1)

t = (96.4466) (19.187) (24.644) (28.046) (5.01)

R = 0.99980 R^2 = 0.99959 STD = 0.55136

F (overall) = 46800 Significant F = 0.00

F (tabular) at 0.01 and at 0.001 per cent = as above

t (tabular) at 0.01 and at 0.001 per cent = as above v - Site (6), (Christchurch, London).

$$S.F. = X_1 + 1.18X_2 + 1.67X_3 + 1.99X_4 + 0.32X_5 + 1.03X_6 + 1.23X_7$$

$$F = (12937)\ (1168)\ (2970)\ (3421)\ (49.5)\ (826)\ (1257)$$

$$t = (113.7)\ (34.2)\ (54.5)\ (58.5)\ (7.04)\ (28.7)\ (35.5)$$

$R = 0.99994$ $R^2 = 0.99989$ STD = 0.27016

F (overall) = 118203.25 Significant F = 0.000

F (tabular) at 0.01 and at 0.001 per cent = as above

t (tabular) at 0.01 and at 0.001 per cent = as above

The computed results and the corresponding "F" and t-test values obtained from statistical tables indicated that the computed passenger car unit values at all sites are statistically highly significant.

Table 2.8.3-13 shows summary of the results, and table (2.8.3-14) shows the average passenger car unit values which were obtained at the six sites investigated.

The computed values are almost the same in all sites. They show the effect of vehicle type in the comparison between the results of straight-ahead only and lanes of straight-ahead and left-turning traffic.

Vehicle Type	Passenger Car Unit Values (standard errors are given in brackets)					
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Passenger Car C	1.00 (0.0139)	1.00 (0.0197)	1.00 (0.023)	1.00 (0.020)	1.00 (0.03)	1.00 (0.022)
Light Goods Vehicle LGV	1.113 (0.033)	1.097 (0.049)	1.05 (0.042)	1.16 (0.037)	1.00 (0.062)	1.18 (0.05)
Heavy Goods Vehicle HGV	1.70 (0.039)	1.79 (0.048)	1.76 (0.045)	1.72 (0.038)	1.63 (0.071)	1.67 (0.036)
Articulated Goods Vehicle & Buses AGV & B	2.10 (0.0373)	1.90 (0.057)	2.01 (0.043)	2.07 (0.038)	1.97 (0.073)	1.99 (0.037)
Motor Cycle MC	0.347 (0.047)	0.56 (0.072)	0.30 (0.054)	0.41 (0.047)	0.30 (0.059)	0.32 (0.047)
Passenger Car (turning left) C_L	1.12 (0.036)	1.12 (0.056)	1.02 (0.053)	1.13 (0.045)	- -	1.03 (0.039)
Light Goods Vehicle (turning left) LGV_L	1.207 (0.049)	1.20 (0.079)	1.18 (0.069)	- -	- -	1.23 (0.037)

Table 2.8.3-13 Summary of PCU Values for the six sites.

Type of vehicle	P.C.U. value
Passenger cars	1.00
LGV	1.10
HGV	1.71
AGV & buses	2.01
MC	0.37
C_L	1.08
LGV_L	1.20

Table 2.8.3-14 Average passenger car unit values at all sites.

Because of the wide range of data obtained at the six sites, the results of the passenger car unit values could be considered as a valuable representation of the traffic composition at the signalized intersections.

2.9 - Comparison of results with existing values

2.9.1 - Saturation flows

The average saturation flow values for a number of studies together with average values obtained for this research study are listed in table (2.9.1-1).

Saturation flow values obtained in this study were based on observation of straight ahead and left-turning traffic and the assumption of zero gradient. Lane width is another factor in this study, it was found to have an average width of 3.45m.

The average saturation flows were found to be statistically similar, but higher flows were recorded at Streatham Hill (A23) traffic signal approach, London. At this site saturation flows were found to be higher because the single lane was designated for straight ahead only. The comparison between predicted flow values (55) and the obtained values in this study showed good agreement.

The results of this study shown in table 2.9.1-1 and the recent results of the Transport and Road Research Laboratory (57), indicate that saturation flows at traffic signal approaches increased during the last decade. This is due to high vehicle manoeuvring performance and the general improvement in intersection design.

STUDY	Saturation Flow PCU/H	Total Lost Time (secs.)
This Study	1887	2.19
TRRL	1940	1.35
Branston	1778	1.50
MV	1902	0.25
Miller	1710	2.10
Southampton	1980	1.52
Sheffield	1568	2.30

TABLE 2.9.1-1

Comparison between studies of saturation flows for single lanes containing unopposed traffic and lost time.

2.9.2 - Mixed turning movement lanes

There has been a limited amount of work reported elsewhere on the effect of turning traffic on saturation flow. This applies especially to lanes containing un-opposed mixed turning traffic and the effect of vehicle type on this parameter. Left-turning movement of passenger cars and light goods vehicles were investigated in this research study at all lanes containing straight-ahead and left-turning unopposed movement.

The effect of a left-turning passenger car was found to be equivalent to 1.08 PCU when turning left and the effect of light goods vehicles were found to be equivalent to 1.20 PCU when turning left.

Average car-to-car headway turning left during saturation flow periods with their precentages are shown in Figure 2.9.2-1 This shows an increase in headway values as their percentages increase in the total flow. Also it was noted that turning-radius had a great effect in reducing saturation flows at signalized junctions. The effect of turning-radius on traffic turning left had a greater effect on buses, heavy commercial vehicles and articulated goods vehicles which in some cases completely blocked the junction while turning. The effect of light goods vehicles turning left at traffic signal approaches

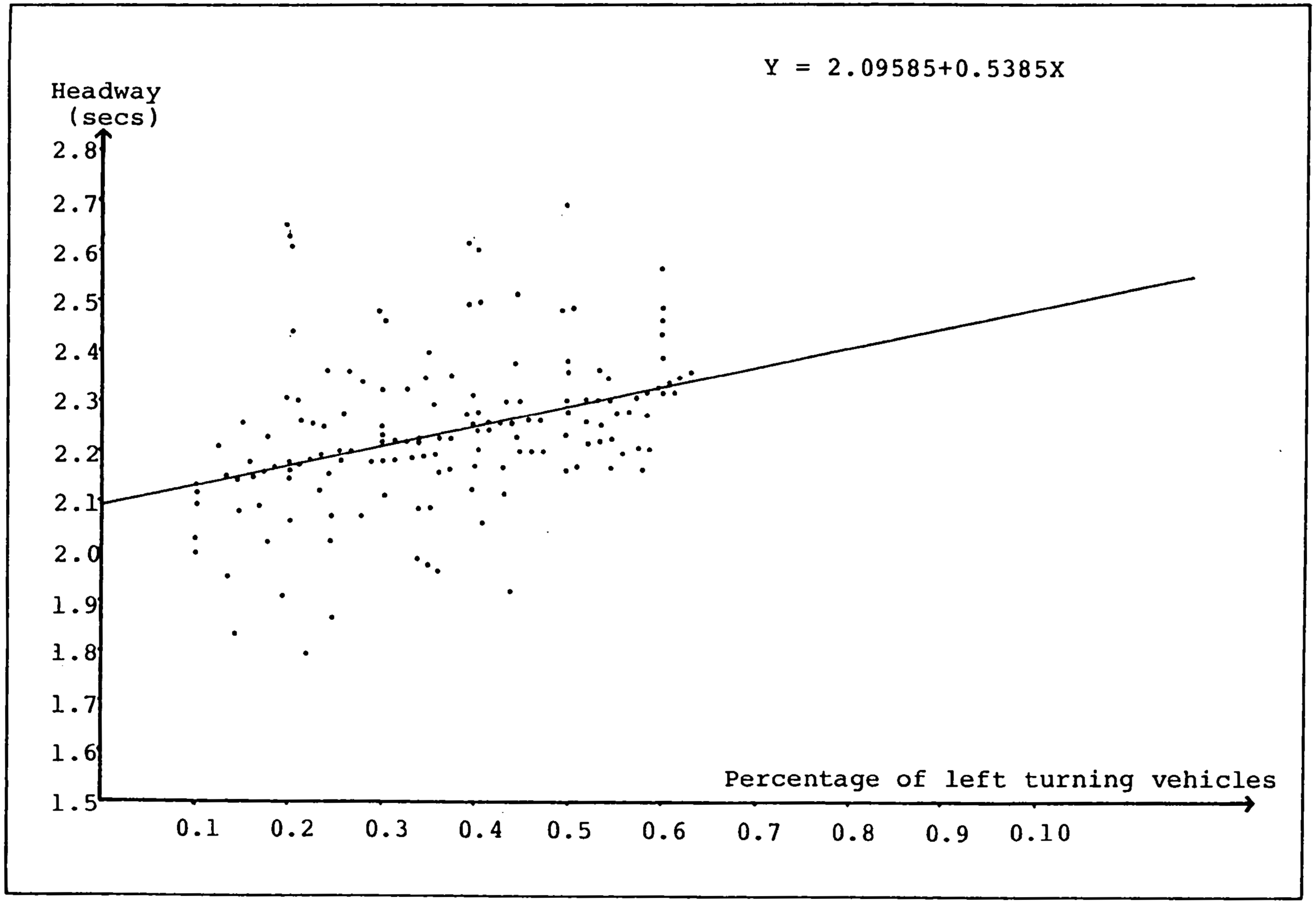


Figure 2.9.2-1 Average Car to Car Headway per Standard Green Time Against Percentage of Left Turning Vehicles.

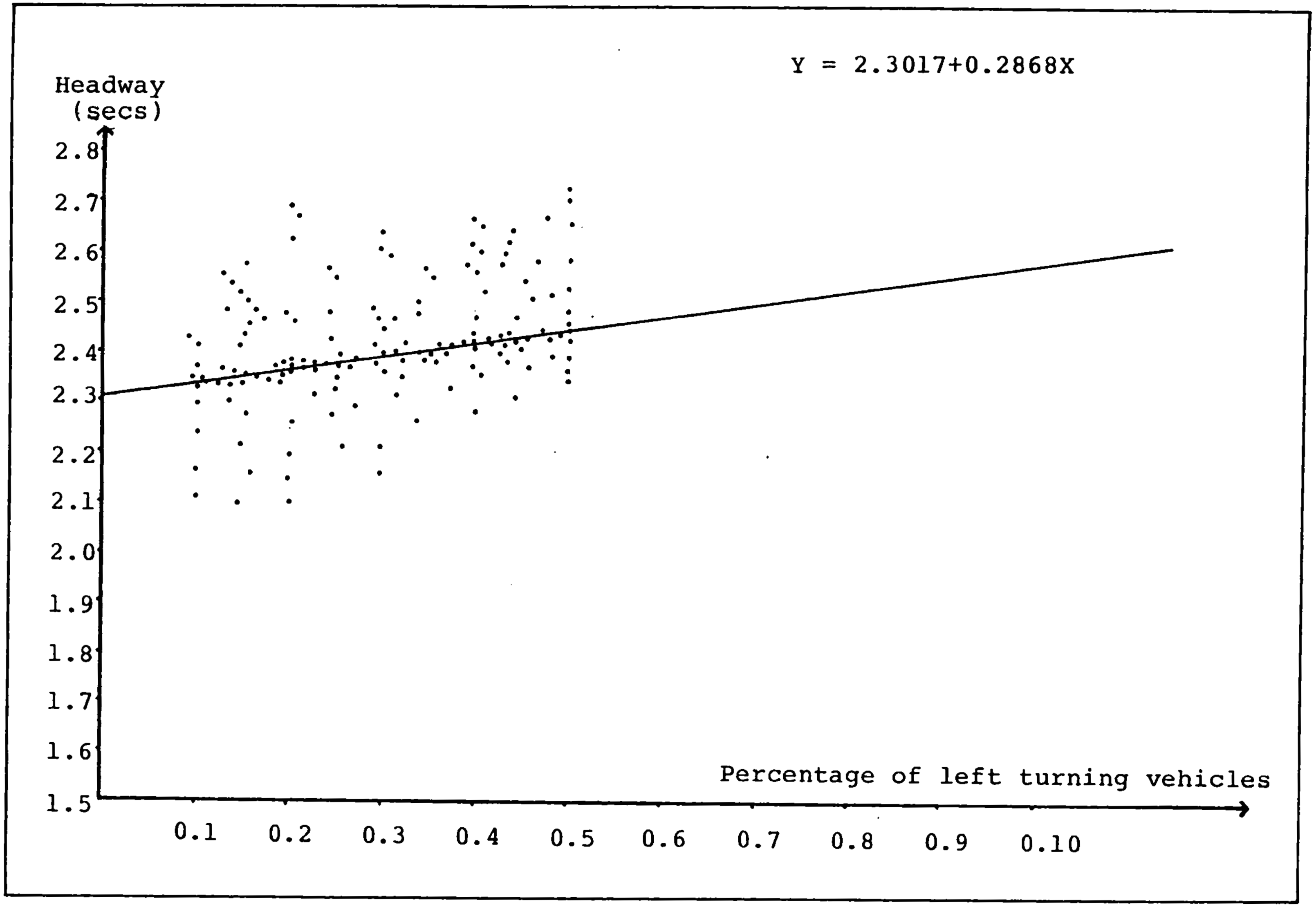


Figure 2.9.2-2 Average Light Goods Vehicle to Light Goods Vehicle Headway per Standard Green Time Against Percentage of Left Turning Vehicles.

were found to be higher than those of passenger cars.

Average light goods vehicle-to-light goods vehicle headway during saturation flow periods with their percentages are shown in Figure 2.9.2-2 which shows a higher value in headway than those found for passenger cars. A positive relationship between average headway (the inverse of saturation flow) and the proportion of turning vehicles was detected. The regression line through the data illustrates that headways increase - or saturation flow decreases - as the percentages of turning traffic increases.

2.9.3 - A Comparison of Computed and Existing Passenger Car Unit Values.

Passenger car unit studies have been conducted by several researchers such as Webster (25), who introduced passenger car units for five types of vehicles which include the following:-

1 Goods vehicle (heavy or medium)	1.75 PCU
1 Bus	2.25 PCU
1 Light goods vehicle	1.00 PCU
1 Motor Cycle	0.33 PCU
1 Pedal Cycle	0.20 PCU

The effects of motor cycles and pedal cycles on saturation flow at traffic signals was studied by Holroyd (64), who introduced the following PCU equivalents:-

1 Motorcycle, moped or scooter	.33 PCU
1 Pedal cycle	.20 PCU

Archer (83) carried out a study of traffic flow through several signal controlled junctions in the London area to find out the effect of right and left-turning traffic on the level of saturation flow through the junction.

He studied a total of six approaches at 5 intersections and he concluded the following PCU equivalents:-

Right turns	1.75
Left turns	1.25

The Road Research Laboratory (53) had suggested the use of the following pcu equivalents when measuring saturation flow at a signalized intersection:-

Heavy or medium goods vehicle	1.75
Bus	2.25
Tram	2.50
Light goods vehicle	1.00
Motorcycle	0.33
Pedal cycle	0.20
Left-turning	1.25 (if in excess of 10%)
Right-turning	1.75

Hart (78) considered the average time gap for each type of vehicle as an indication for the pcu equivalences of the vehicle. From his research on 32 approaches of signalized intersections he found out the following results:-

Light commercial	1.00 PCU
Heavy commercial	1.40 PCU
Truck with trailer	2.50 PCU
Bus	1.80 PCU
Bicycle	0.13 PCU

He stated that his observations were insufficient to find the left and right-turning equivalence.

Leong (55) in his study in Australia on pcu equivalences introduced the following relationship for S:-

$$S = 1700 - 12CV - 11RT - 2LT$$

where

S = saturation flow/lane (veh/hr)

CV = % commercial vehicles

RT = % right-turning vehicles

LT = % left-turning vehicles

and hence he deduced that:

Commercial vehicles:	Each 1% reduces saturation flow by 0.70% (range of CV studied 0-19%).
Right-turning vehicles:	Each 1% reduces saturation flow by 0.65% (range of RT 0-42%).

Left-turning vehicles: Each 1% reduces the saturation flow by 0.12% (range of LT 0-88%).

Branston and Van Zuylen (61) used Multiple Linear Regression to obtain pcu values of vehicle types and turning movement at one intersection which they studied in London.

The following values were quoted in their study using the synchronous and the asynchronous methods:

	Synchronous	**Asynchronous**
Van	1.28	1.14
Bus	1.61	1.79
Commercial	1.59	1.74
Motorcycle	0.08	0.04
Left-turning	0.89	0.89

The asynchronous method was recommended because it is not biased and needs no correction. The low pcu value of left-turns was due to many left-turning movements at the intersection studied.

Miller (52) studied the passenger car unit equivalences in Australia using headways and assuming that the headways after the fourth vehicle has crossed the stop-line were constant. His studies showed that a commercial vehicle is equivalent to 1.85 but for simplicity he recommended the use of 2.0 pcu's. Left-turning movement is equivalent to 1.25 whereas right-turning movement has a pcu equivalence of 2.9. Branston (79) found that pcu values tended to increase as saturation flow increased at a site. This implied that within the capabilities of their vehicles, drivers of commercial vehicles and buses maintained the same time headway of the vehicle in front irrespective of traffic and site conditions. The following values were quoted in his study as "typical values" as a result of his analysis:-

Bus	1.70
Commercial (medium)	1.33
Commercial (heavy)	1.70

Pedal cycles and motorcycles had very little effect on saturation flow if their proportion was less than 20% and so they could be ignored. For percentages greater than 20, the pcu of motorcycles was 0.15 and that of pedal cycles was 0.10. As for left-turners, Branston found that if the site layout was such that straight on cars could overtake a left-turning car in the junction, the pcu value was generally 1.0. If not then the pcu value was on average 1.33. Branston's sample sizes ranged between 700 and 2700 vehicles for the different types studied.

Joubert (80), in Southampton showed that the pcu equivalence of a commercial vehicle was 1.34 and that of a bus was 1.76, whereas a motorcycle was 0.81 pcu.

White showed the following results:-

Commercial vehicle	1.71 PCU
Two-wheeler	0.59 PCU

In the study by McDonald (82) in which 14 signal controlled junctions in Southampton were examined to determine the relationship between saturation flow and green time, the pcu value for a commercial vehicle was 1.38, for a bus 1.56 and a two-wheeler 0.60.

In a study by the Transport and Road Research Labotatory (84), to find the passenger car equivalent values at signalized intersections for various vehicle types and turning movements, the results were different from those given by RRL (25). The study was mainly for commercial vehicles and buses. The criteria used was the length of the commercial vehicle and these were grouped into four classes according to their length (L):

$L \leq 6m$, $6m<L\leq 9m$, $9m<L\leq 12m$, $L>12m$.

For non-turning traffic, the headways of cars had an average value of 2.4 sec. and hence the PCU equivalent of other types were calculated by comparing their headway with this value.

The average results were:-

	PCU	Av. Length(m)	Sample Size
Cars	1.0	4.5	194
Commercial vehicles:			
$L\leq 6m$	1.1	5.0	38
$6m<L\leq 9m$	1.4	8.1	45
$9m<L\leq 12m$	1.7	10.7	15
$L<12$	2.1	14.0	30
Buses and coaches	1.7	10.0	2
Left-turning	1.1		
Right-turning	1.9		

Table 2.9.3-1 shows a summary of the results for

Study	Commercial Vehicle	Bus	Right turning Vehicle	Left turning Vehicle	Motor Cycle	Pedal Cycle
RRL (25)	1.75	2.25	1.75	1.25 if>10	0.33	0.20
Miller (52)	2.00	-	2.90	1.25	-	-
Branston & Van Zuylen (61)	1.74	1.79	-	0.89	0.04	-
Hart (78)	1.40	1.80	-	-	-	0.13
Leong (55)	1.70	-	1.65	1.12	-	-
Branston (79)	1.33-1.7	1.70	-	1.33	0.15 if>20%	0.10
McDonald (82)	1.38	1.56	-	-	0.60	-
White (81)	1.71	-	-	-	0.59	-
Joubert (80)	1.34	1.76	-	-	0.81	-
TRRL (56)	1.5-2.3	2.0	-	-	0.40	0.20

Table 2.9.3-1 Passenger Car Unit Values by various studies

various studies.

The computed results of passenger car unit equivalences obtained from the analysis of data from several signalized intersections in London and West Yorkshire are shown in Tables 2.8.3-1 to 2.8.3-12. They were found to be similar and consistent.

The recommended values which represent the average values for all the analysis are shown in Table (2.8.3-13).

2.10 Vehicle type delay relationship

The various theoretical analyses dealing with delay to traffic flows at traffic signal intersections have used mathematical expressions for the average delay per vehicle. The factors which are included in these expressions are: saturation flow, cycle time, proportion of the cycle effectively green and vehicles' arrival. Some expressions which were suggested by Newell (35) and Miller (34) use the variance-to-mean ratio of the number of arrivals per cycle (I-ratios).

To show vehicle-type effect on delay traffic signals, Webster's full expression is used in this study.

The expression is:-

$$d = \frac{c(1 - \lambda)^2}{2(1 - \lambda x)} + \frac{x^2}{2q(1 - x)} - 0.65 \left(\frac{c}{q^2}\right)^{1/3} x^{(2+5 \lambda)}$$

in which;

C = the cyle time in seconds,

g = the effective green time in seconds,

λ = the proportion of the cycle that is effectively green, i.e $\lambda = g/c$,

q = the arrival rate in vehicles/sec.,

s = the saturation flow in pcu/sec.,

x = the degree of saturation; i.e. $x = q/s$.

The main varying factor in the expression for the analysis is the degree of saturation.

$$\text{Saturation flow} = \frac{1}{\bar{hi}}$$

where $\bar{hi}$ is the mean headway for vehicles leaving the stop line.

Saturation flow increases as $\bar{hi}$ decreases. It was found from observed data at the six sites that this value for passenger cars is the smallest. Mean time headway for all vehicle types considered during the analysis (passenger cars, light goods vehicles, heavy goods vehicles, buses/articulated goods vehicles and motor-cycles) are calculated. They are:-

Vehicle Type	Mean time headway (seconds)					
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Passenger cars	1.76	1.96	1.63	1.74	3.34	2.42
Light goods vehicles	1.96	2.15	1.71	2.02	2.34	2.86
Heavy goods vehicles	2.99	3.51	2.87	2.99	3.82	4.05
Buses and articulated goods vehicles	3.70	3.72	3.28	3.60	4.61	4.81
Motor-cycles	0.61	1.1	0.49	0.71	0.70	0.77

The mean headway values are used to calculate saturation flow for a particular type of vehicle. Webster's full expression was then used to calculate average delay to vehicles using fixed parameters for signal setting, 60 seconds for cycle time and 30 seconds for effective green time. Also seven traffic-flow values were used ranging between 200 v/h and 500 v/h.

Average delays are calculated for combined traffic using the saturation flow values obtained from observed data, as discussed in section 2.8.2 of this chapter.

Tables 2.10-1 to 2.10-6 show average delay values for the six sites under consideration. These results are plotted against traffic-flow values and are shown in figures 2.10-1 to 2.10-6.

The figures show the variation in delay values between various types of vehicles. The following results were obtained:-

(i) For low traffic values the difference in average delays are not as significant as those for higher values of traffic flows. This indicates that higher percentages of heavy commercial vehicles and buses suffer higher delay under the same conditions available.

Traffic flow V/h	Average delay (seconds)					
	100% Cars	100% LGV	100% HGV	100% AGV	100% M.C.	Combined traffic
200	8.73	8.94	10.37	11.68	7.81	8.89
250	9.09	9.37	11.31	13.22	7.89	9.31
300	9.47	9.83	12.39	15.23	7.97	9.74
350	9.87	10.32	13.70	18.37	8.06	10.21
400	10.29	10.84	15.43	25.02	8.14	10.71
450	10.75	11.41	18.13	50.77	8.23	11.25
500	11.23	12.04	23.49	-	8.32	11.84

Table 2.10.1 Computed average delay values using Webster's expression. (Site 1 results).

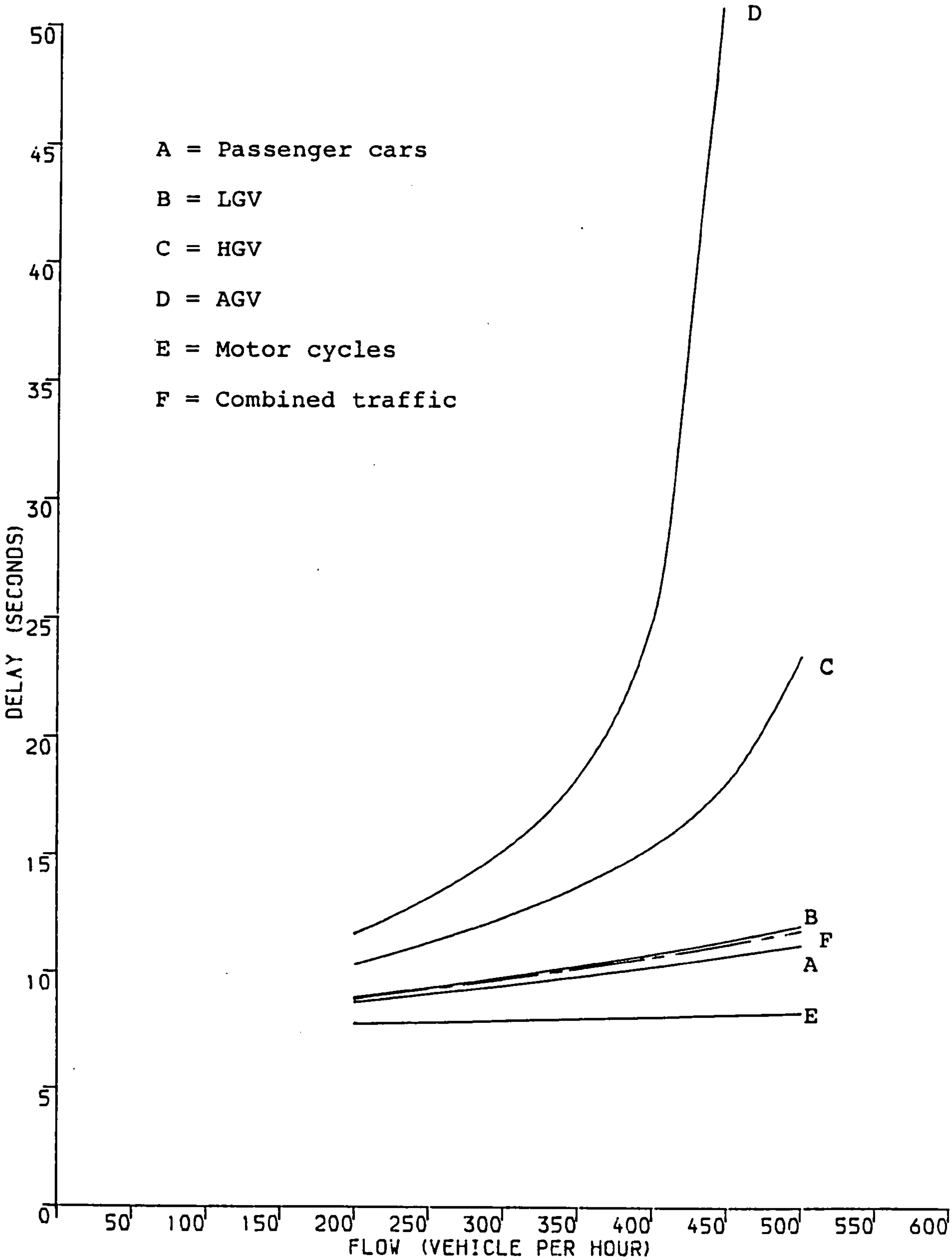


Figure 2.10-1 (Site 1) A comparison between average delay values using Webster's expression.

Traffic flow	Average delay (seconds)					
V/h	100% Cars	100% LGV	100% HGV	100% AGV	100% M.C.	Combined traffic
200	8.94	9.16	11.31	11.74	8.14	8.79
250	9.37	9.67	12.66	13.31	8.31	9.17
300	9.83	10.21	14.35	15.37	8.49	9.57
350	10.32	10.79	16.76	18.65	8.68	10.00
400	10.84	11.43	21.12	25.78	8.88	10.45
450	11.41	12.13	32.78	55.60	9.08	10.94
500	12.04	12.95	141.13	-	9.29	11.46

Table 2.10.2 Computed average delay values using Webster's expression. (Site 2 results).

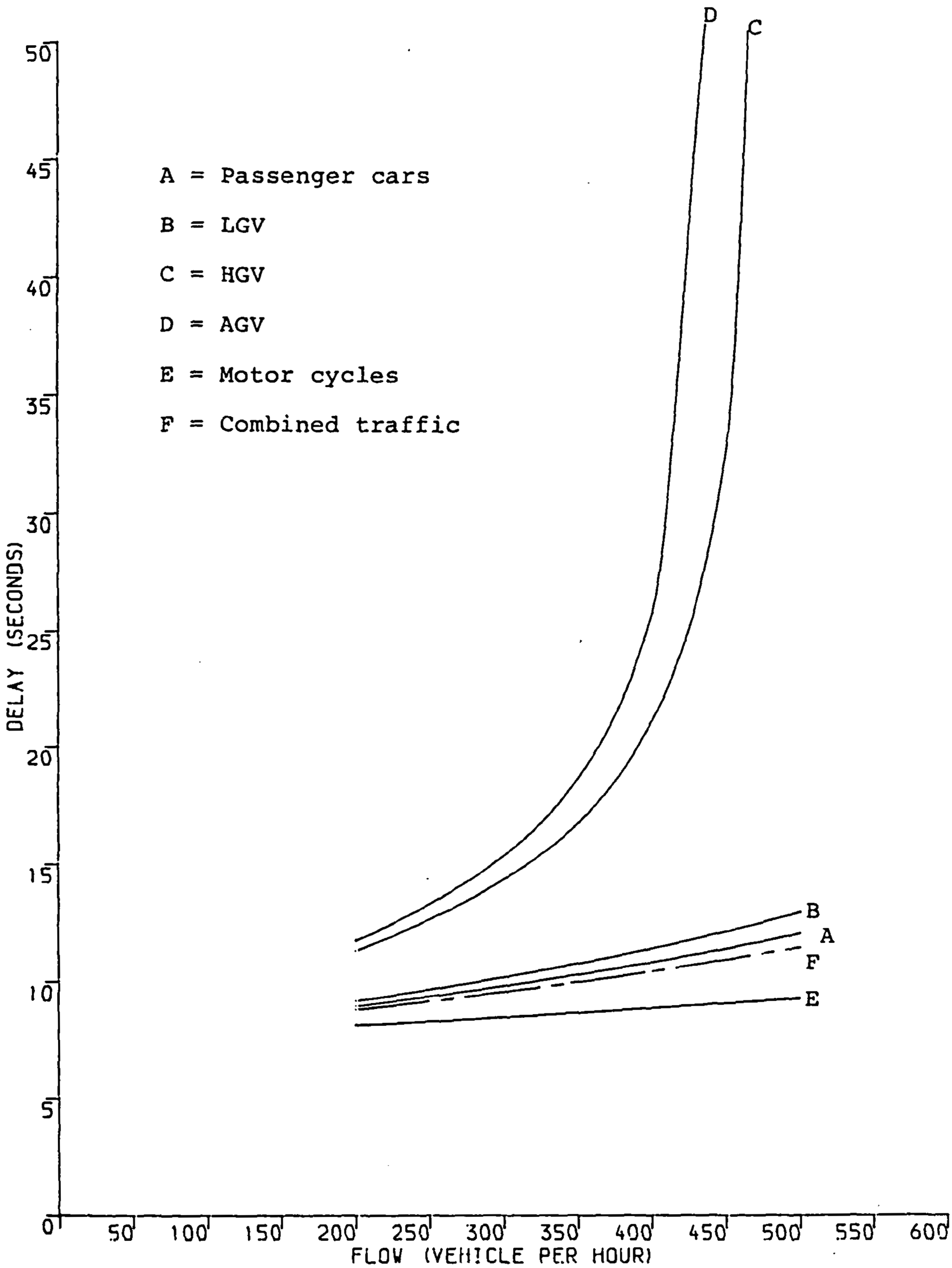


Figure 2.10-2 (Site 2) A comparison between average delay values using Webster's expression.

Traffic flow	Average delay (seconds)					
V/h	100% Cars	100% LGV	100% HGV	100% AGV	100% M.C.	Combined traffic
200	8.60	8.68	10.17	10.86	7.74	9.47
250	8.92	9.02	11.04	12.02	7.80	9.96
300	9.25	9.38	12.02	13.39	7.86	10.48
350	9.60	9.77	13.17	15.19	7.93	11.04
400	9.97	10.17	14.63	17.95	7.99	11.66
450	10.36	10.60	16.74	23.45	8.06	12.34
500	10.78	11.06	20.41	41.00	8.12	

Table 2.10.3 Computed average delay values using Webster's expression. (Site 3 results).

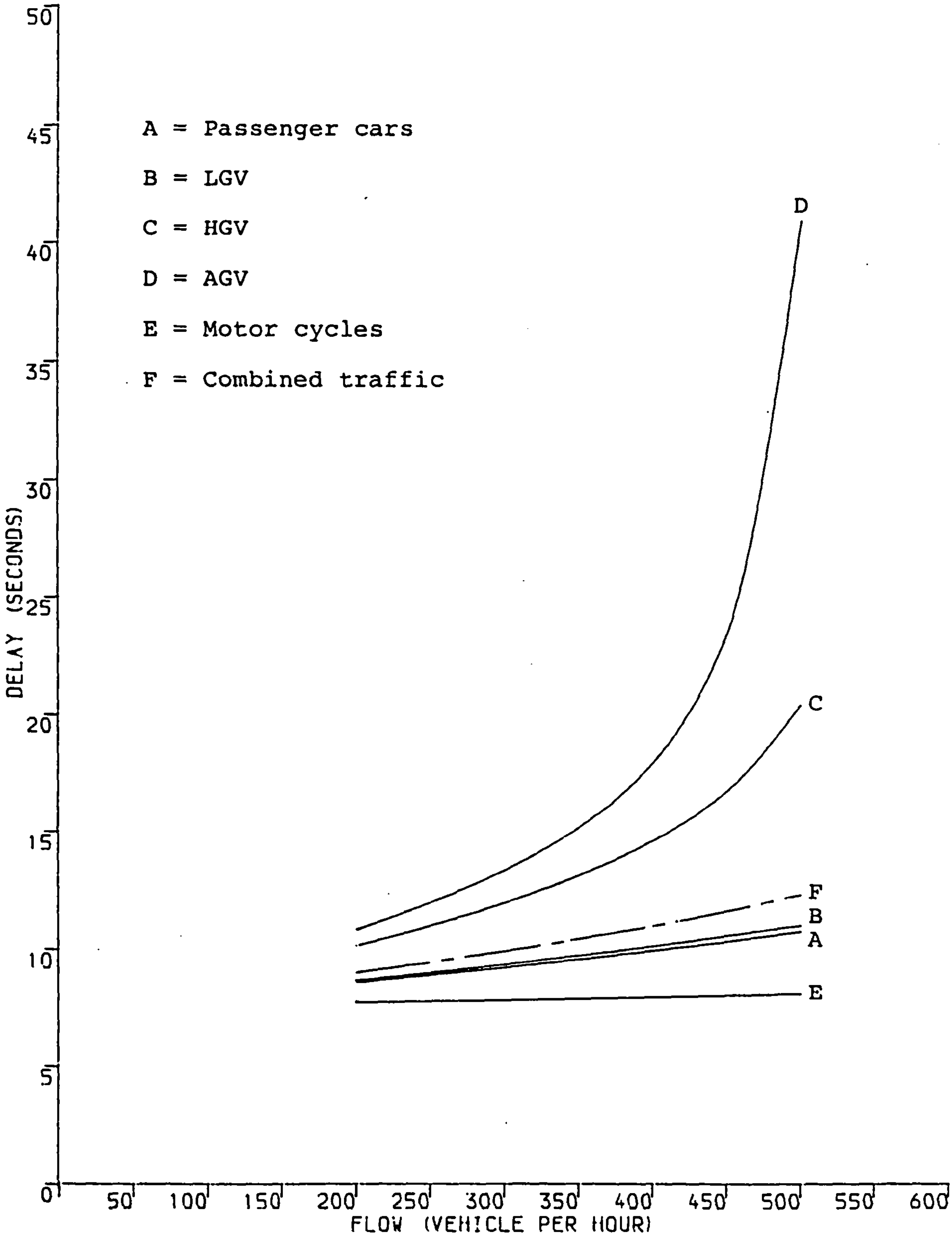


Figure 2.10-3 (Site 3) A comparison between average delay values using Webster's expression.

Traffic flow	Average delay (seconds)					
V/h	100% Cars	100% LGV	100% HGV	100% AGV	100% M.C.	Combined traffic
200	8.71	9.01	10.37	11.49	7.87	9.04
250	9.06	9.46	11.31	12.93	7.97	9.50
300	9.43	9.94	12.39	14.76	8.07	9.99
350	9.82	10.46	13.70	17.50	8.17	10.52
400	10.24	11.02	15.43	22.81	8.28	11.09
450	10.68	11.62	18.13	39.43	8.38	11.72
500	11.15	12.30	23.49	60.19	8.49	12.42

Table 2.10.4 Computed average delay values using Webster's expression. (Site 4 results).

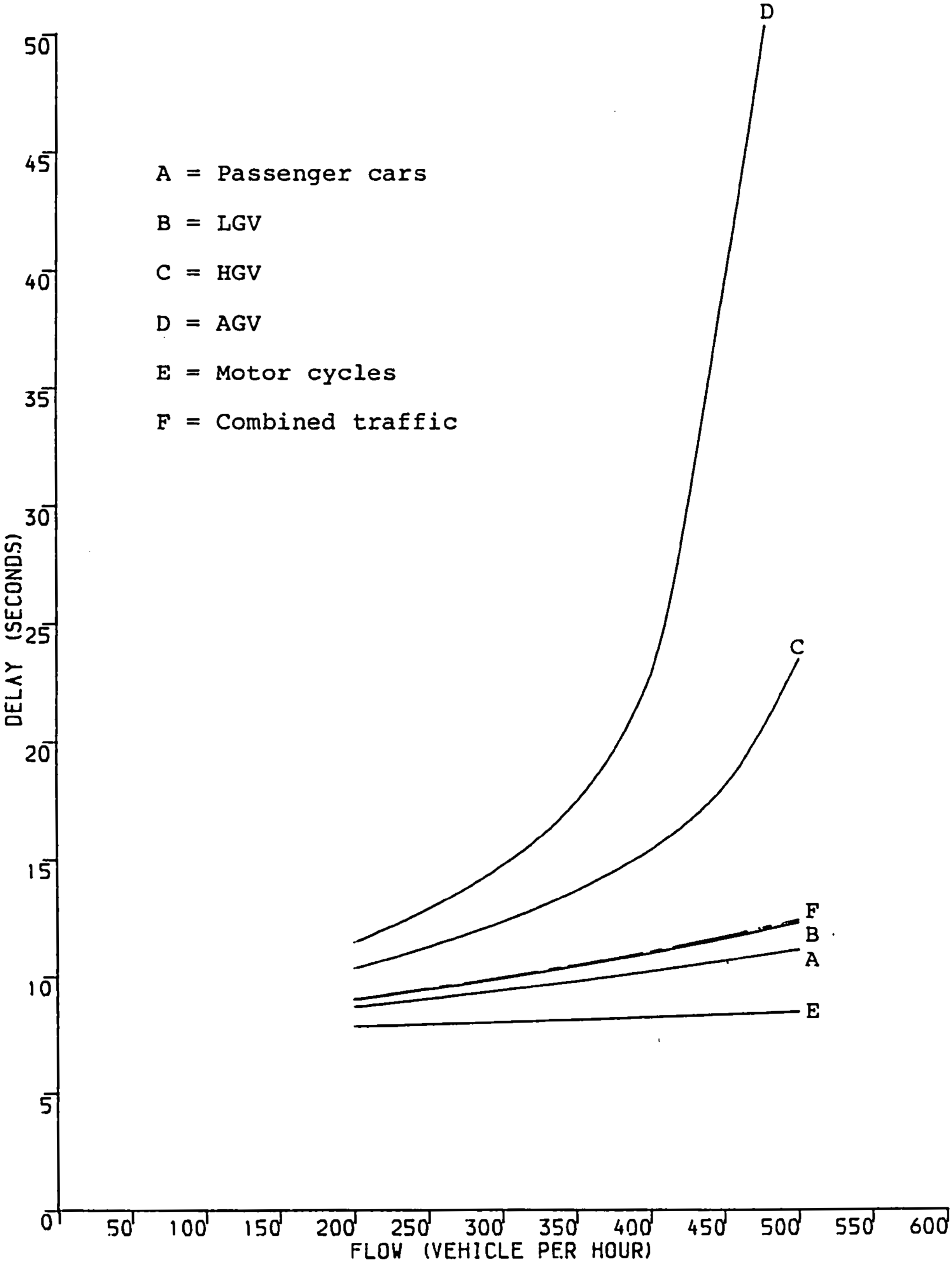


Figure 2.10-4 (Site 4) A comparison between average delay values using Webster's expression.

Traffic flow	Average delay (seconds)					
V/h	100% Cars	100% LGV	100% HGV	100% AGV	100% M.C.	Combined traffic
200	9.40	9.40	11.96	14.06	7.86	8.78
250	9.99	9.99	13.64	17.20	7.96	9.15
300	10.62	10.62	15.93	23.37	8.06	9.54
350	11.32	11.32	19.77	45.97	8.16	9.96
400	12.10	12.10	29.10	-	8.26	10.41
450	13.00	13.00	85.62	-	8.37	10.89
500	14.10	14.10	-	-	8.47	11.40

Table 2.10.5 Computed average delay values using Webster's expression. (Site 5 results).

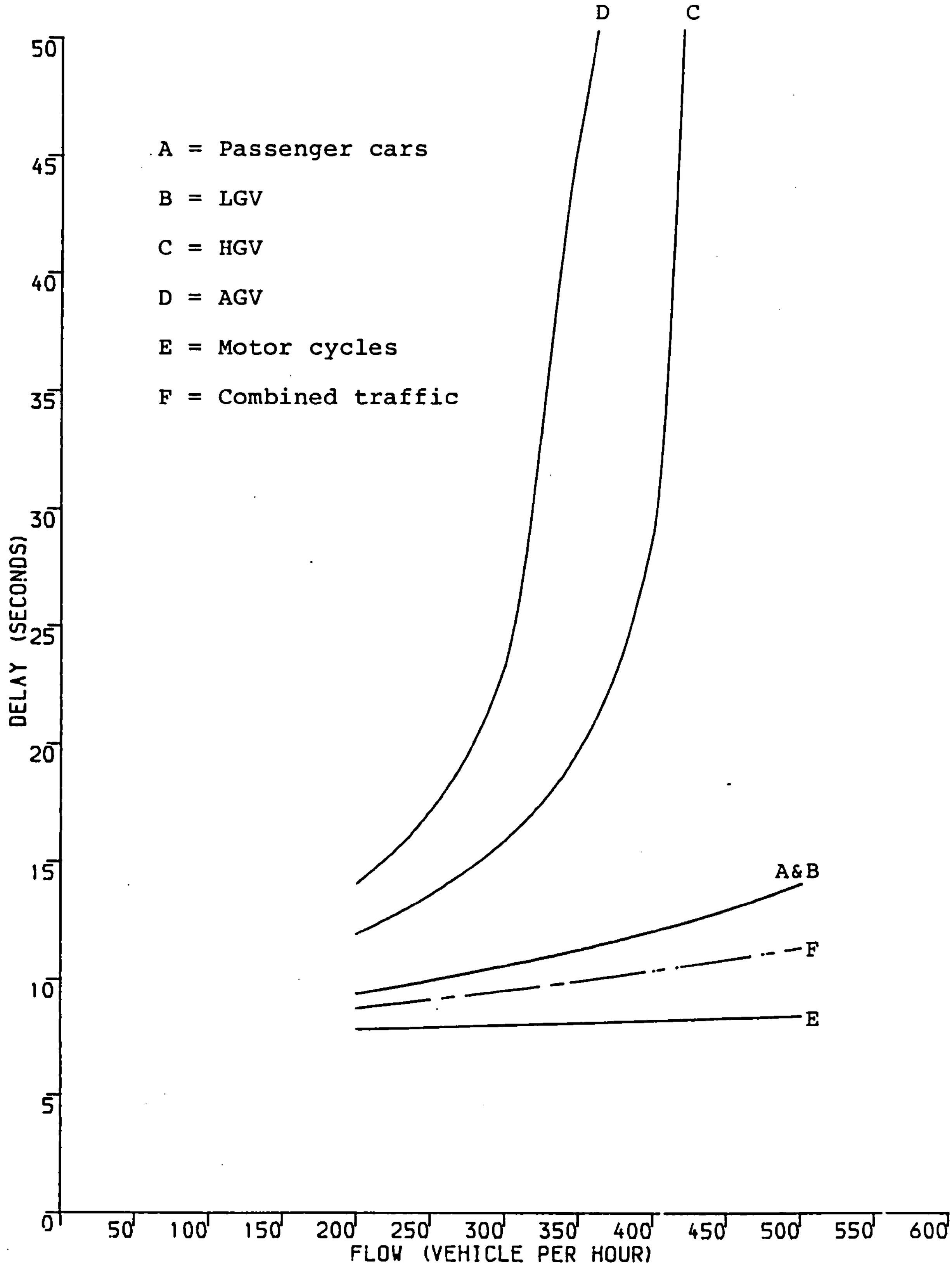


Figure 2.10-5 (Site 5) A comparison between average delay values using Webster's expression.

Traffic flow V/h	Average delay (seconds)					
	100% Cars	100% LGV	100% HGV	100% AGV	100% M.C.	Combined traffic
200	9.51	10.14	12.51	14.71	7.90	8.83
250	10.13	11.00	14.51	18.47	8.02	9.21
300	10.81	11.97	17.45	26.98	8.13	9.63
350	11.56	13.10	23.25	74.16	8.25	10.07
400	12.40	14.53	43.10	-	8.36	10.54
450	13.41	16.57	-	-	8.48	11.04
500	14.68	20.07	-	-	8.61	11.58

Table 2.10.6 Computed average delay values using Webster's expression. (Site 6 results).

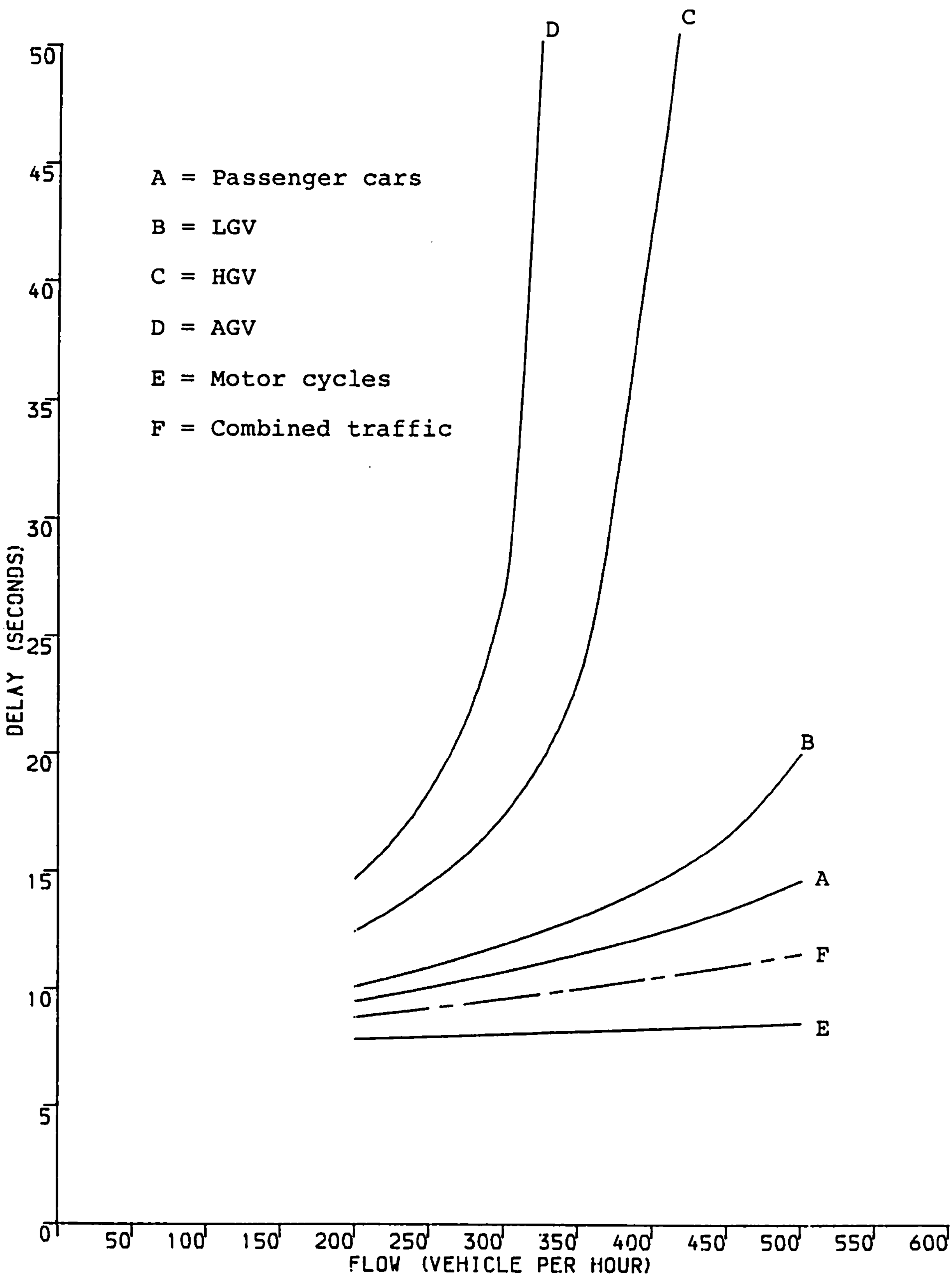


Figure 2.10-6 (Site 6) A comparison between average delay values using Webster's expression.

(ii) The variation in average delay values are mainly due to the effect of traffic composition i.e. percentages of heavy commercial vehicles, as the geometrical features were kept the same for each site in the calculation.

(iii) The results of average delay for the combined traffic flows are closer to those of the smaller vehicles (passengr cars). This is because of their higher percentages present in the traffic flow.

3

Effects of Vehicle Type at Roundabout Entries

CHAPTER THREE

Effect of vehicle type at roundabout entries

3.1 - Introduction

The roundabout is a very widely used form of intersection. Basically it is a one-way carriageway around a central island. The analysis of its operational characteristics must consider the effect of its geometric design in conjunction with traffic factors. The behaviour of traffic at each approach is related to the composition of arriving traffic and general driver characteristics.

Capacity of the intersection is affected by vehicle type, especially buses and articulated vehicles with their lower acceleration ability and other operating characteristics such as lag and gap acceptance requirements. In order to obtain a standardised flow rate a conversion procedure should be applied. In a previous British method (85), the following factors were used to obtain the passenger car equivalents at roundabout approaches:

1	Bus	2.8 PCU
1	Heavy and medium commercial vehicle	2.8 PCU
1	Light commercial vehicle	1 PCU
1	Motor cycle, moped or scooter	0.75 PCU
1	Pedal cycle	0.5 PCU

In recent years, much research has been devoted to capacity and delay estimation. The capacity of a roundabout can now be predicted entry by entry (86), and the balance of inflows for the whole roundabout calculated by a method which treats a roundabout as a series of T-junctions linked by a common circulating carriageway. The unified formula (38) for the entry capacity now allows this method to be put on a general basis for design and assessment.

3.2 - Historical review

3.2.1 - Roundabout intersections

The principles of the roundabout operation were discussed by Watson and Dawson (87) 1936. They examined the capacity of roundabouts by considering the number of vehicles which could pass through a single weaving point, taking into account the physical dimensions of the vehicles, their speed of travel and the angle of convergence of the intersecting traffic streams. From these analyses they concluded that the theoretical capacity of a single merge was between 1,200 and 1,800 vehicles per hour depending on the initial assumptions.

The Transport and Road Research Laboratory 1955 (88) decided to carry out experiments on a large scale using an artificial layout to find the capacity of a single weaving section. The experiments were carried out at Northolt Airport where different traffic parameters and roundabout shapes could be tested. A continuous flow of vehicles was passed through isolated weaving sections by recirculating vehicles to allow steady state flows to occur with queues on all entries, covering variation in the proportion of weaving traffic and composition of traffic.

The Transport and Road Research Laboratory, the

University of Sheffield and the consultants Freeman Fox and Associates (86) have carried out extensive work to study the performance of roundabouts. Their work showed conclusively that the original Wardrop formula for the capacity of roundabout "weaving" sections, as incorporated in pre-1975 Department of the Environment design manuals, was no longer accurate under present traffic conditions. It over-predicted significantly and the "proportion weaving" term did not appear to explain the fluctuations in observed capacity either at individual sites or between sites.

The relationship between saturated entry flow "F" and the corresponding controlling circulating flow "fc" was investigated in order to develop a completely new formula. They found that "fc" was significantly less than the values implied in both the original and the revised Wardrop formula, and the new equations were much more successful than the Wardrop formula at predicting the within-sections variation of Qe with Qc.

The capacity entering flow, Qe was expressed in the form:

$$Qe = F - fc\ Qc \qquad \text{pcu/h}$$

where $$F = 233e_1 \left(1.5 - \frac{1}{\sqrt{r_1}}\right) - 255 \quad \text{efu/h}$$

(efu = entry flow units)

$$fc = 0.0449\ (2e_1 - w) + 0.282$$

The PCU value of a heavy vehicle is taken as 2.0, and the ranges of the parameters in the data used were:

e_1 4.0 to 12.5 m

$e_1/\sqrt{r_1}$ 0.74 to 3.30

$2e_1 - w$ - 2.5 to 9.50 m

Qc 580 to 3890 pcu/h

The Transport and Road Research Laboratory suggested that these equations were unlikely as they stand to represent the final solution for design purposes. However, further development was made to check the formulae against data available for other sites and their experimental work introduced a unified approach to all roundabout design. The final form of the equations are discussed in section (3.3.1) of this chapter.

3.2.2 - The priority rule

In the late 1950's and during the Transport and Road Research Laboratory experiments, the operation of roundabout intersections was based on the concept that no particular traffic stream had the "right of way". However, with a large flow entering a roundabout, the right of way, once established, tends to remain with the flow and forces traffic already in the circulating section to wait for suitable gaps in the entering flow. As traffic on the roundabout built up it tended to interfere with other entry flows and often produced a locking condition. The capacity then dropped temporarily to a very low value because very few vehicles were able to pass through the roundabout until it was clear. Long queues often started to build up on the approaches during this low flow period, but the traffic on all approaches once more forced its way into the roundabout when the congestion had been removed and immediately locked it up again.

In order to reduce the possibility of locking, the "give way to traffic from the right" priority rule was introduced using signs and carriageway markings similar to those at conventional priority type intersections (89, 90). In all cases, it was immediately observed that these priority signs virtually eliminated locking and gave an increase in capacity compared with previous forms of control.

The priority-to-the-right rule at roundabouts was introduced in Britain in November 1966 by the Ministry of Transport and notice of this announcement and advice on its implementation was given to highway authorities in Road Circular No. 27/66 (91).

The introduction of this new rule brought a fundamental change in the mode of operation of roundabouts and meant that the concept of weaving is no longer relevant.

3.3 - Roundabout traffic parameters

3.3.1 - Traffic capacity

All new schemes must be justified by cost benefit analysis and it is therefore useful to be able to predict capacities, queues and delays for a wide range of circumstances.

The formulae proposed by Kimber (1980) (92) have been used for several years to calculate capacity of single-island roundabouts of all types and on an entry by entry basis. In 1981, the Department of Transport released a computer program, ARCADY (Assessment of Roundabout Capacity and Delay) which was developed at the Transport and Road Research Laboratory and which implemented these formulae.

The roundabout is treated as a series of entries linked by a common carriageway. It was suggested that capacity is linearly related to the circulating flow and for each entry the capacity, Qu, is calculated using the following equation:

$$Qe = F - fc\ Qc \quad \text{pcu/h}$$

where F and fc represent the intercept and slope respectively of the entry/circulating flow relationship and are given by:

$$F = 303\ X_2 K$$

$$fc = 0.210 \text{ to } K(1+0.2X_2)$$

where

$$K = 1 - 0.000347\ (\phi-30) - 0.987\ ({}^{1}/_{r}-0.05)$$

$$tD = 1 + 0.5/\left(1+\exp\left(\frac{D-60}{10}\right)\right)$$

$$X2 = V + \frac{e-V}{1-25}$$

and $$S = 1.6\ \left(\frac{e-V}{1'}\right)$$

where V is the approach road half-width (m)

- e entry width (m)
- ϕ entry angle (degree)
- r entry radius (m)
- 1' the length of flare (m)
- D the inscribed circle diameter (m)

These geometric parameters are described in more detail in figure (3.2.4-1). The acceptable ranges of parameters for use in ARCADY2 are:

- V : 2.2 - 12m
- e : 3 - 16m
- ϕ : 0 - 18°
- r : > 3m
- 1' : any
- D : > 13m

"F" and "fc" were derived from extensive observations on the public roads and on the Transport and Road Research Laboratory test track, and apply to small and large roundabouts alike.

Equation (3.2.4-1) is modified if the entry is grade separated according to observations made at this type of site ( ), and becomes:

$$Qe = 1.11F - 1.40\ fc\ Qc \quad \text{pcu/h}$$

Since the circulating flow crossing one arm is dependent on the flow entering from the other arms, an interactive procedure is used within each time segment for calculating the entry capacities. Initially it is assumed that there is circulating flow past the first entry, so that the flow is taken as either the demand flow itself or the value F, whichever is the lower. This entry flow, after subtraction of these vehicles taking the next exit, becomes the circulating flow past the next entry, enabling that entry capacity to be calculated from its entry/circulating flow relationship. The flow from the entry is then equal to the demand flow or the capacity, whichever is the smaller. Thus the circulating flow past the next entry can be calculated, progressing around the roundabout. When, after a complete cycle the circulating flow past the first entry is calculated,a revised entry flow can be determined as the starting point of a second iteration and the whole process repeated. After several iterations, the entry flows from each arm converge to their final values.

It was found that this process is a convenient way of solving the set of 'n' simultaneous equations (n = number of arms) of the forms of equation (3.2.4-1), where the Qc values are each functions of the turning proportions and entry flows from the other arms.

Unless the system is in equilibrium, the queue lengths at the start of a time segment will be different from those at the end of a segment. The flow actually entering the roundabout from any arm in a segment will therefore be equal to the demand flow plus the initial queue length minus the final queue length. This leads to a second level of iteration being necessary because the first level cannot take account of the actual queue lengths remaining and thus over-estimates the circulating flow levels. Before the second level of iteration can take place, queue lengths must be calculated by using time-dependent queueing theory.

3.3.2 - Gap acceptance studies

The behaviour of drivers at a priority highway intersection when entering a major road from a minor road is complex. They have to make decisions with skill in order to provide for safe crossing or merging with a traffic flow in the major road. The analysis of traffic behaviour requires the introduction of two time intervals in the analysis. The first is a "gap" which is the time interval between the arrival of a major-road vehicle at the intersection and the arrival of the next major road vehicle. The second is a "lag" which is the time interval between the arrival of a minor road vehicle at the intersection and the arrival of the next major road vehicle.

The usual hypothesis for minor road drivers' behaviour at a priority intersection is the time hypothesis. It is assumed that gap or lag acceptance occurs when a minor road driver makes a precise estimate of the arrival time of the approaching major road vehicle.

The driver is said to accept a lag when entering the intersection before the major road vehicle reaches it. But if the same driver waits until the major road vehicle has passed before entering the intersection, he is said to reject the lag, so the driver must wait for an accepted gap. The minor road driver can reject many gaps but he accepts one lag or gap each time he enters the intersection. Driver decision to accept or reject a lag or a gap presented to him on arrival at the intersection is affected by various factors:

- junction layout and in particular entry approach visibility,
- the difference in vehicle type operational characteristics,
- entry traffic flow
- the composition of traffic flow
- the flow in the major road.

Several research projects have been conducted to study the gap and lag characteristics of at-grade intersections. In these investigations various techniques were used to analyse intersection flow patterns under different roadway and traffic conditions.

An extensive study was carried out in the U.S.A. by Greenshields (93), for both controlled and uncontrolled intersections to investigate the time intervals accepted by drivers when entering the major traffic flow. In particular, stop sign controlled intersections were included in these investigations. According to his study the average minimum acceptable gap was defined as that value which is accepted by fifty per cent of the drivers.

The concept of the "critical lag" was investigated by Raff and Hart (94) in which the number of accepted lags shorter than the critical lag is equal to the number of rejected lags longer than this specific value. Four intersections were included in the study which gave values for the critical lags of 4.6, 4.7, 5.9 and 6.0 seconds. The variation between these four results could be influenced by sight obstruction, major road speed, major road width, and the traffic flow in the minor road.

The technique of probit analysis was used by Solberg and Oppenlander (95) in the statistical treatment of their observations. In addition, two other methods which were developed by Raff and Bissell (96) were considered in the evaluation. The comparison of the probit, Raff and Bissell methods of the median accepted are shown in table (3.3.2-1) below:

Method	Combined Lag and Gap (second)		
	Right turn	Left turn	Straight
Probit	7.36	7.82	7.18
Raff	7.45	7.85	7.35
Bissell	7.35	7.65	7.10

Table 3.3.2-1 Comparison of the probit values.

The acceptance distribution are well described by a linear relationship between the probit of acceptance and the logarithm of acceptance time. There is no significant difference between the median gap acceptance time at the intersections under study. However, significant variation was found between right and left turning drivers and straight through drivers.

Ashworth (97), has proposed that where the gap acceptance is normally distributed the biased acceptance curve is displaced from the true curve by an amount S^2q, where "q" is the major road volume (veh/sec) and "S" is the standard deviation of the driver gap acceptance curve. The value of "S" may be determined by probit analysis techniques or, alternatively, estimated from the cumulative gap and lag acceptance curve using the relation,

S = (85 per cent gap - 15 per cent)/2.07.

Ashworth's mathematical model was based on the assumption that the traffic in the major road is randomly distributed and the minimum gap acceptance for the minor road drivers follows a normal distribution curve, and the gap acceptance is normally distributed with a mean "M" and standard deviation "S".

Salter (98), has investigated the first decision driver gap and lag acceptance at a number of priority highway intersections in the West Riding of Yorkshire. The observed data were fitted to cumulative normal, cumulative log-normal and exponential distributions and it was noted that the best fit was obtained to a cumulative log-normal distribution.

3.3.3 - Delay criteria

Delay at roundabout may be taken as the extra travel time imposed on vehicles whilst moving through the intersection. This delay was stated by Webster and Newby (90) to consist of the following two components;

1 - Delay caused by slowing down to negotiate the roundabout, by travelling the extra distance and by accelerating to normal speed.

2 - Delay caused by intersection with other vehicles using the roundabout (i.e., delay from queueing and weaving action).

For the first component, Webster and Newby gave 9 seconds for a normal travel speed of 48 km/h (30 mph), 12.5 seconds for speed of 64 km/h (40 mph), and 17 seconds for 80 km/h (50 mph), they observed that roundabout diameter had little effect on the delay.

Their findings for the second component is no longer valid because of the introduction of new parameters in delay calculation by the Transport and Road Research Laboratory. In their report (37), the geometric delay at non-signalized intersections was investigated. The report draws together the results from studies conducted during the last few years by Southampton University, in which roundabouts, major/minor priority junctions and large at-grade and grade separated intersections have been investigated.

The geometric delay for a specific movement at roundabout is given by:

$$\left(\frac{V_A - JS}{a_{AB}}\right) + \left(\frac{V_D - JS}{a_{CD}}\right) + \frac{d_{DB}}{JS} - \left(\frac{d_1 + d_{AB}}{V_A}\right) - \left(\frac{d_2 + d_{CD}}{V_D}\right) \quad \text{sec.}$$

where V_A, V_D are the approach and departure speeds respectively, measured at points where speeds are not influenced by the junction (m/s)

JS is the speed within the junction (m/s)

a_{AB} is the deceleration rate approaching the junction and is given by $a_{AB}=1.06(V_A\_JS)/V_A+0.23$ m/s²

a_{CD} is the acceleration rate leaving the junction and is given by $a_{CD}=1.11(V_D-JS)V_D+0.02$ m/s²

d_{BC} is the distance travelled within the junction (m)

d_1, d_2 are the distances between the centre of the junction and the entry and exit respectively (m)

d_{AB} is the distance over which deceleration towards the junction takes place and is given by,

$$d_{AB}=(V_A{}^2-JS^2)/2\ a_{AB}\ \text{m}$$

d_{CD} is the distance over which acceleration away from the junction takes place and is given by,

$$d_{CD}=(V_D{}^2-JS^2)/2\ a_{CD}\ \text{m}$$

For a left turn,

JS = 0.84 (ER +   EXR) m/s

where ER, EXR, are the entry and exit kerb radii (m)

For a straight-ahead movement where 0.5 (ENA+EXA)>20°;

$$JS = 0.9 \quad D + 2.03 \text{ m/s}$$

where D is the inscribed circle diameter.

The geometric parameters are defined more fully in figures (3.3.3-1) and (3.3.3-2).

If the calculated $JS > V_A$, then $JS = V_A$ and $d_{AB} = 0$ and $d_{CD} = 0$.

If $JS > V_A$ and $JS > V_D$, then $JS = \frac{1}{2} (V_A + V_D)$.

If delay calculated is <0, then delay = 0.

For movements at a grade-separated roundabout from a grade-separated arm to a non grade-separated arm, the geometric delay is reduced (by a factor of 1.2) because of higher acceleration rates.

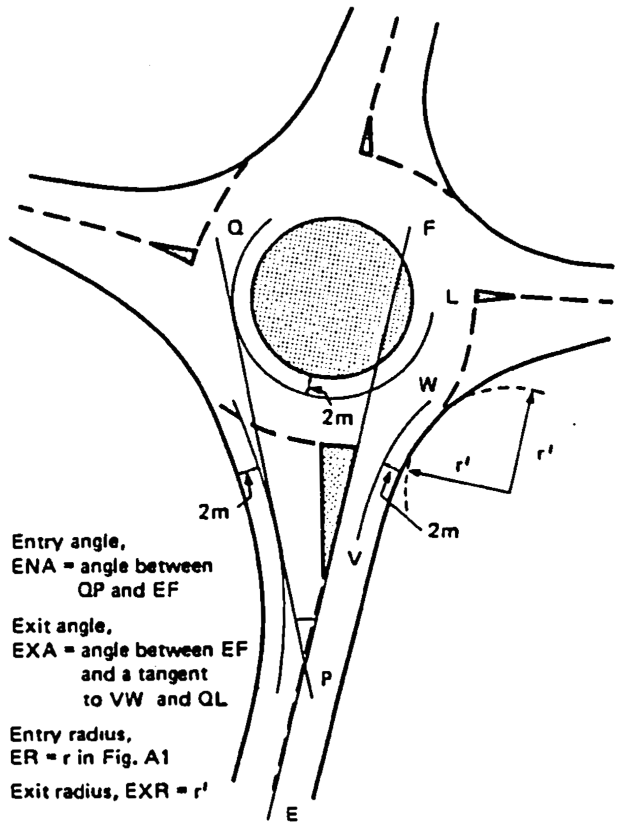


Figure 3.3.3-1 The measurement of parameters associated with geometric delay.

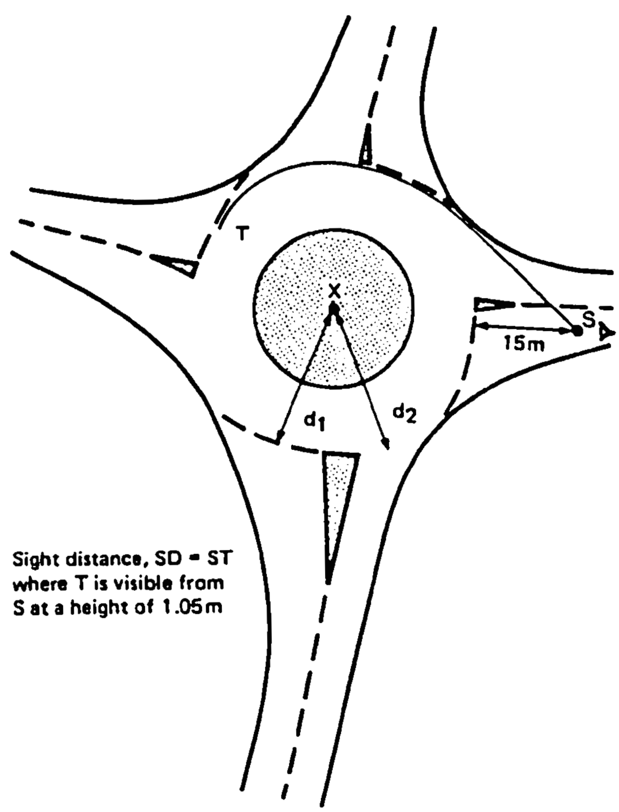


Figure 3.3.3-2 The measurement of parameters associated with geometric delay.
(Reproduced from reference No. 38)

3.4 - An analysis of vehicle type effect on junction performance

3.4.1 - Vehicle type and circulating flow

At roundabouts the circulating carriageway traffic is mainly affected by the intersection geometric features such as size and number of lanes. Flared approaches and smaller central islands enable higher capacities to be achieved for a given junction area than had previously been possible.

In a report by the Transport and Road Research Laboratory (99), the passenger car unit values for heavy commercial vehicles were found not to have a wide variation between entry and circulating flows. In contrast with other vehicles, mainly motor cycles and cycles, there was a wide difference. This was because they often move between queues of traffic, taking little extra road space, whereas in the circulation they inhibit entering vehicles almost as much as light vehicles do. Other types of vehicles,especially articulated goods vehicles and double decker buses,have greater impact on driver behaviour and decision when entering or leaving the circulation carriageway. This is because of their turning manoeuvres within the circulating carriageway, their low speeds and their tendency to block the view of car drivers so that they have to wait until the road is clear to be able to decide on whether to accept or reject gaps on the circulating carriageway. These effects cause greater delays to traffic flow at roundabouts intersections.

3.4.2 - Vehicle type and roundabout performance

The capacity of a roundabout is defined as the total flow which can enter the intersection when there is saturation (queueing) on all the approaches. The calculation of circulating flow can be obtained from the entry relationships and turning movements of the remaining entries.

In a report by the Transport and Road Research Laboratory (99), the effect of traffic composition is expressed in passenger car units for two types of vehicles as follows;

	Entry	Circulating
Heavy vehicles	1.9	1.7
Other vehicles (two-wheelers mainly motor cycles)	0.2	0.80

From these PCU values it is obvious that the range of values is relatively narrow, bearing in mind that there is often considerable variability in composition analysis of public road data.

The composition between the present passenger car unit values reported by Semmens (100)and the previous values reported by Glen, Summer and Kimber (99) indicate that entry circulating flows are treated the same when expressed in PCU's, while in the previous report they were treated differently, especially for two-wheelers.

In the study of entry/circulating flow relationship, gap acceptance criteria has received much attention. This is because it represents the degree of vehicle-vehicle interactions that take place in the region of the entry. The effect of vehicle type on gap acceptance will be investigated in section 3.7 of this chapter.

3.5 - Site selection and description

3.5.1 - Introduction

A wide range of data is required to study traffic performance at roundabout intersections, which will represent various sites' characteristics. The sites chosen for the study were required to have one or more entries saturated for 40 minutes during either morning or afternoon peak periods or both.

In all counts, vehicles were classified into four categories which are:

- passenger cars;
- light goods vehicles:
- heavy goods vehicles;
- articulated goods vehicles, buses and coaches.

Data was collected from three conventional roundabout approaches, two at Hackney, Greater London, and one at the M606-motorway terminating roundabout at Bradford.

3.5.2 - Site selection criteria

Sites chosen were to be saturated for a period of forty minutes at least during peak periods and were to carry a range of different types of vehicles in order that their passenger car unit values could be simultaneously calculated, and vehicle type effect on junction performance be investigated.

The intersections were to be 'conventional' in layout, so comparison with other results would be possible. The entries were required to have good visibility with no obvious sight restrictions (e.g. buildings, vegetation, parked vehicles), nor any significant number of pedestrian movements. In addition, bus stops and similar obstructions were not to be located immediately beyond the exit to the intersections.

Given suitable weather conditions, the flows at all approaches were heavy enough to allow sufficient data to be collected in order to derive gap acceptance functions.

3.5.3 - Site descriptions

The following sites were chosen based on the aforementioned criteria.

1 - Old Street (A5201)/City Road (A501) roundabout, Hackney - London

The intersection was a four-leg conventional roundabout. It was an ideal situation for the investigation because of its location on two major routes, the A5201 and the A501. Both routes carried a large proportion of heavy commercial vehicles and double-deck buses serving the central commercial areas in Greater London, see figures (3.5.3-1) and (3.5.3-2).

The approach considered at this intersection was the east-bound section of Old Street (A5201), which was the inner city ring road. The approach consisted of a two-lane carriageway each 5.8m wide. The near side lane was level with good visibility and clear road markings at the entry, and no pedestrian interference. Saturation level was reached during both the morning and afternoon periods between 7.30 a.m. and 9.30 a.m. and 2.30 p.m. and 6.00 p.m. respectively.

2 - City Road (A501)/Old Street (A5201) roundabout, Hackney - London

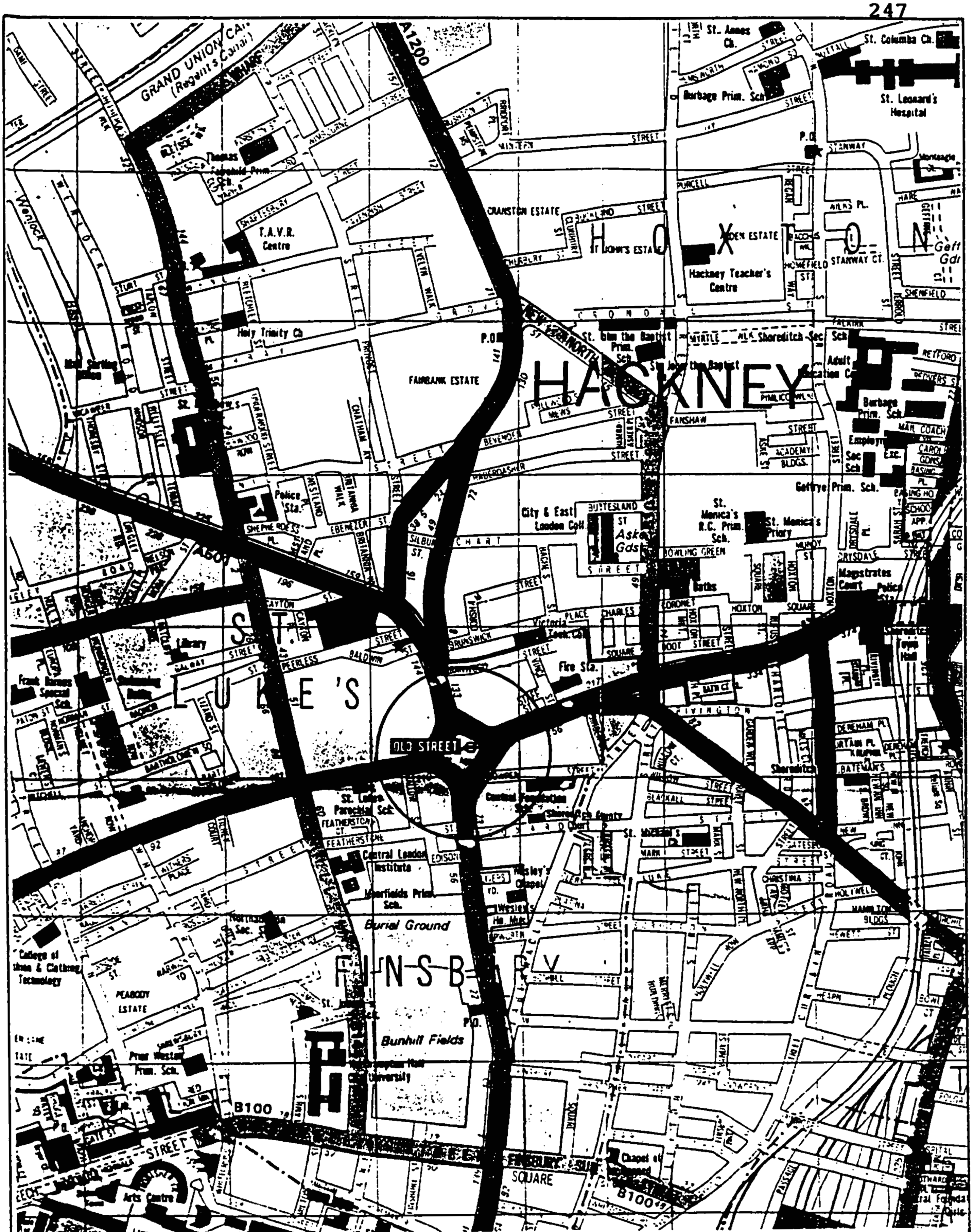


Figure 3.5.3-1 (A5201) Old Street/City Road Roundabout - LONDON.

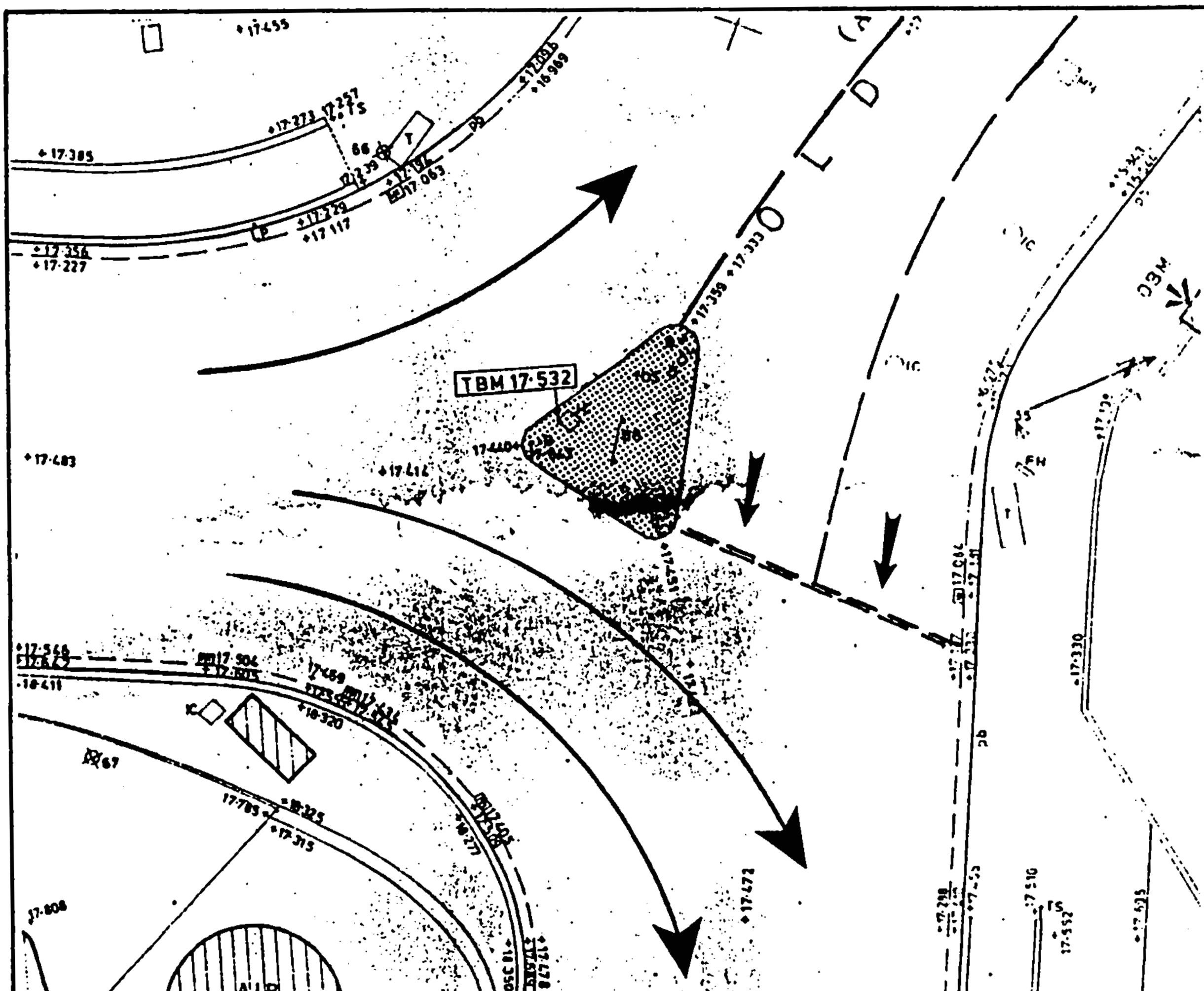


Figure 3.5.3-2 (A5201) Old Street approach - London.

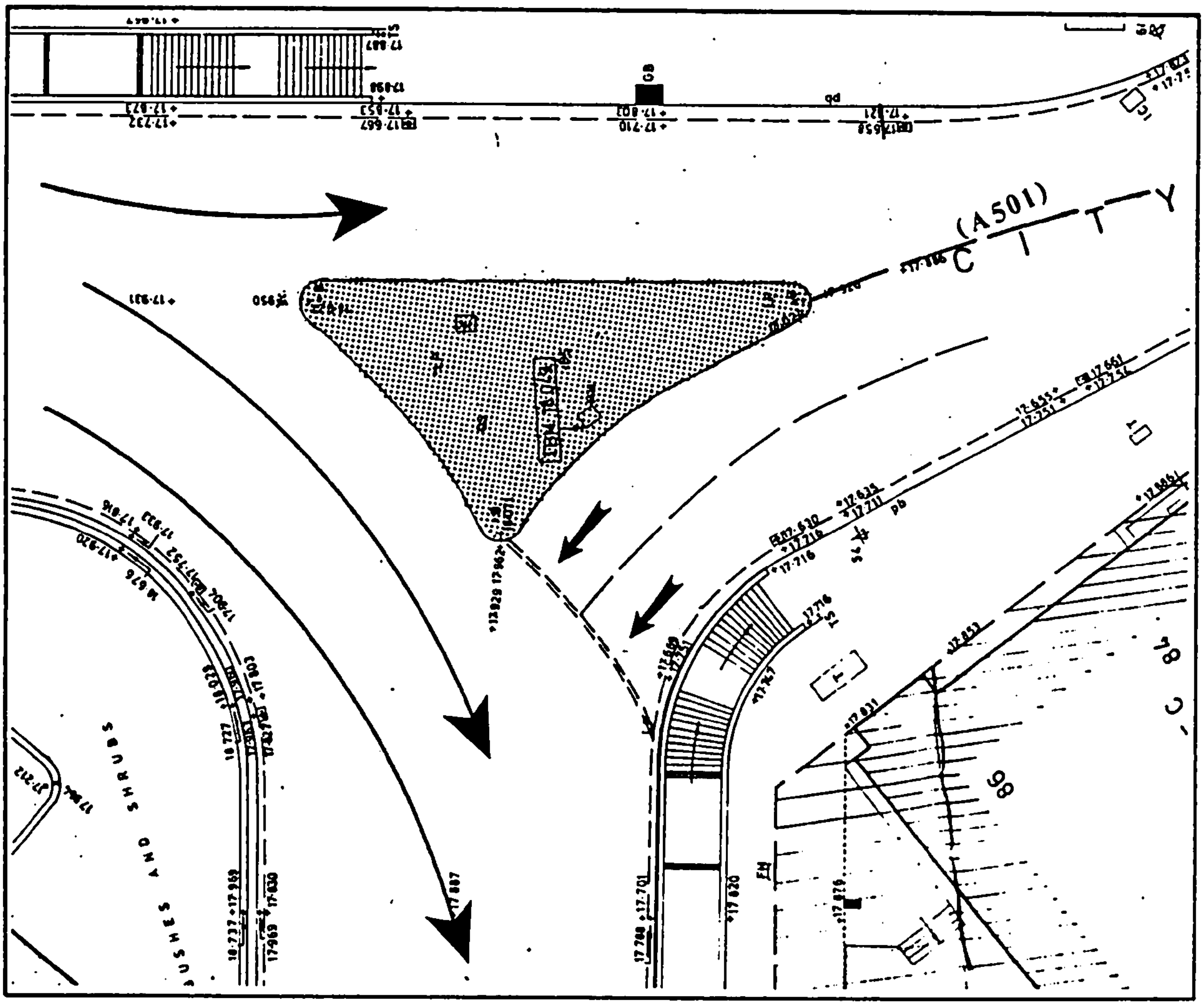


Figure 3.5.3-3 (A501) City Road approach - London).

The approach under consideration was the second leg of the previously described Old Street roundabout. It represented the main route connecting east and central London. It was of two lanes each 4m wide, see figures (3.5.3-1) and (3.5.3-3). Saturation level was reached at the approach during both morning and afternoon periods between 7.30 a.m. and 9.30 a.m. and 2.30 p.m. and 6.0 p.m. respectively.

Filming of this approach and the approach previously discussed in (1) was carried out using a video filming technique. The camera was mounted on the central island of the roundabout which is part of Old Street underground station. It was accessible using the subway to the central island of the roundabout

The near side lane used for the observation was almost level and there was no pedestrian interference at the point of entry.

A summary of the dimensions of the approaches and the circulating carriageway is given in table (3.5.3-1).

3 - <u>M606/A6036 Rooley Lane roundabout, Bradford</u>

This intersection was situated south of Bradford in the West Riding of Yorkshire, at the termination of the

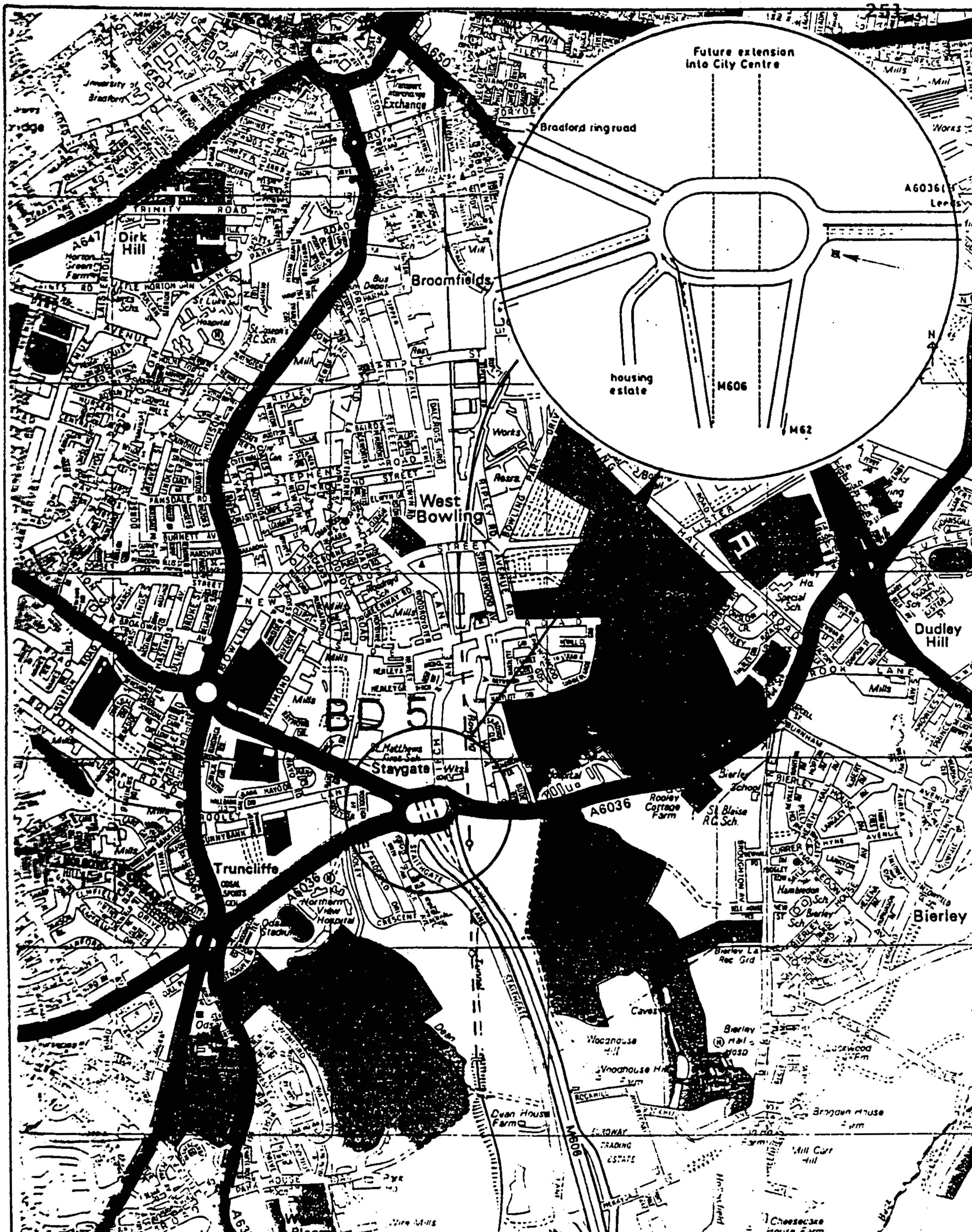


Figure 3.5.3-4 (M606/A6036) Rooley Lane Roundabout - BRADFORD.

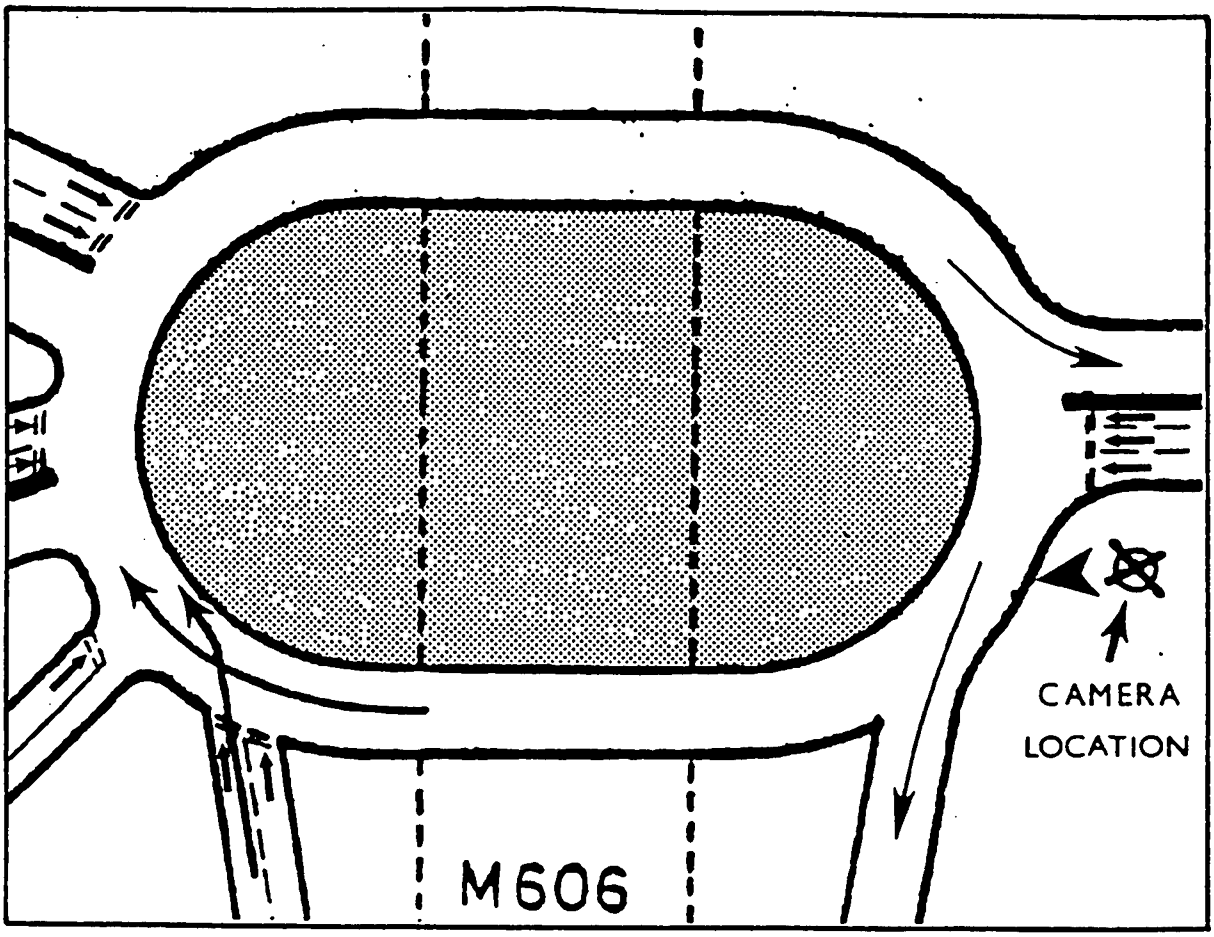


Figure 3.5.3-5 (M606/A6036) Rooley Lane Roundabout - BRADFORD.

M606 Motorway. The approach considered for this study was the west-bound carriageway of the M606 which represents the end of the link between the M62 Motorway and Bradford city centre, see figures (3.5.3-4) and (3.5.3-5).

The approach carried a large proportion of heavy commercial vehicles from the M62 Motorway to Bradford. The other important route connected at this intersection is the A6036, which is a dual carriageway in both east and west directions at the roundabout. It carried traffic between Halifax and the northern and western parts of Leeds; it also forms part of the City of Bradford ring road.

During peak hours, there were continuous traffic queues in both lanes of the approachbut the queues on the near side lane were of longer duration. The duration of saturation condition was long enough for the purposes of this study during both morning and afternoon peak periods.

Site	Location	Number of Lanes	Circulating Width (m)	Entry Width (m)
Old Street (A5201)/City Road	London	2	11	11.6
City Road (A501)/Old Street	London	2	11	8.0
M606/A6036) Rooley Lane	Bradford	2	11.25	7.5

Table 3.5.3-1 Sites Summary

3.6 - Filming technique and data collection

The video filming technique described in section (2.6.2) of the previous chapter was used to collect data from each site investigated. This technique was a very good source of information for the analysis of traffic flows.

At site (1), see figure (3.5.3-2), the observation was carried on for a total of 10 hours during both the morning and afternoon peak periods.

The traffic movements at both approaches at this site are shown in figures (3.5.3-2) and (3.5.3-3). Both approaches were free from any physical obstructions with clear traffic signs. Also traffic flow was free from pedestrian involvement, with pedestrian subways located on each side of the approach, see figures (3.5.3-2) and (3.5.3-3).

Filming of both approaches was conducted during September, October and November 1984.

At site (2) the observations were carried out for a total of 10 hours of filming, during the morning and afternoon peak periods.

Observation at site (3) was carried out for a period of 15 hours of filming during February, March and April 1984.

Filming of the approach was carried out during morning and afternoon peak periods between 7.30 a.m. and 9.00 a.m. and 2.30 p.m. to 5.30 p.m. respectively. The camera was located on top of a hill at the side of the roundabout as indicated in figure (3.6.3-5).

3.7 - Gap acceptance observation analysis and distribution fitting

3.7.1 - Vehicle type gap acceptance distributions

A gap acceptance analysis was conducted for each of the observed approaches. Single lane traffic movements (near side lane) at the entry were analysed using first drivers' decisions in order to avoid bias towards slower drivers.

The main aim of the analysis of gap acceptance was to study the behaviour of different types of vehicles. Four main types of vehicles were considered (passenger cars, light goods vehicles, heavy goods vehicles and articulated goods vehicles, including buses) in the analysis of entry movements and a comparison between gap acceptance for each type of vehicle was made. This comparison was between the calculated mean values and standard deviation of the observed data for each vehicle type.

The probit-analysis method developed by Finney (101) was used. The theoretical fit of the cumulative distribution is performed by plotting the probit of the percentage acceptance against the lag-class mark. The best straight line fit is then drawn through such points, using the linear-regression technique. The mean gap acceptance is the value when the probit is 5 and the standard deviation of lag acceptance is equal to minus the reciprocal of the slope of the straight line.

Results of the vehicle type gap acceptance analysis are given in tables (3.7.1-1) to (3.7.1-15) and are represented graphically together with their probit transformation in figures (3.7.1-1) to (3.7.1-24).

A summary of the distribution parameters calculated for each site is given in tables (3.7.1-6), (3.7.1-11) and (3.7.1-16). It can be seen from these tables that the correlation coefficients indicate a good fit of the cumulative normal distribution to the classified observed data of lag and gap acceptance.

The results showed variations in gap acceptance for the different types of vehicles under consideration. It was found that the difference between mean gap acceptance values for passenger cars and articulated goods vehicles is significant in urban situations when compared with the semi-rural situation. In the former case the difference between the two values of mean gap acceptance was found to be 3.0 seconds, while in the latter case it was 0.67 seconds. This condition is due to the geometrical features of the semi-rural intersection which provide better facilities for vehicles to manoeuvre, with less interference from other road users.

Vehicle type mean gap acceptance results show a strong relationship between the vehicle type and the criteria of accepting or rejecting a gap or lag. Heavier and longer vehicles (articulated goods vehicles, buses, etc.) due to their lower acceleration capabilities and size compared with passenger cars require longer gaps to enter the circulating carriageway. This effect of the vehicle type performance reflects the high number of gap or lag rejections and the fewer gap and lag acceptances. From summary tables (3.7.1-5), (3.7.1-10) and (3.7.1-15), it can be seen that the average gap accepted by passenger cars is 3.10 seconds, where the average gaps accepted by light goods vehicles, heavy goods vehicles and articulated goods vehicles were found to be 4.14 seconds, 4.64 seconds and 4.92 seconds, respectively.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	0	23	-	-	-
1.5 ---- 2.49	24	150	13.79	3.91	21.2
2.5 ---- 3.49	95	127	42.79	4.82	42.10
3.5 ---- 4.49	101	30	77.10	5.74	69.20
4.5 ---- 5.49	85	8	91.40	6.37	88.20
5.5 ---- 6.49	70	4	94.59	6.60	96.80
6.5+	163	0	100	-	

Table 3.7.1-1 Classification of data and distribution fitting of lag and gap acceptance of entry movement (passenger cars),M606 Motorway.

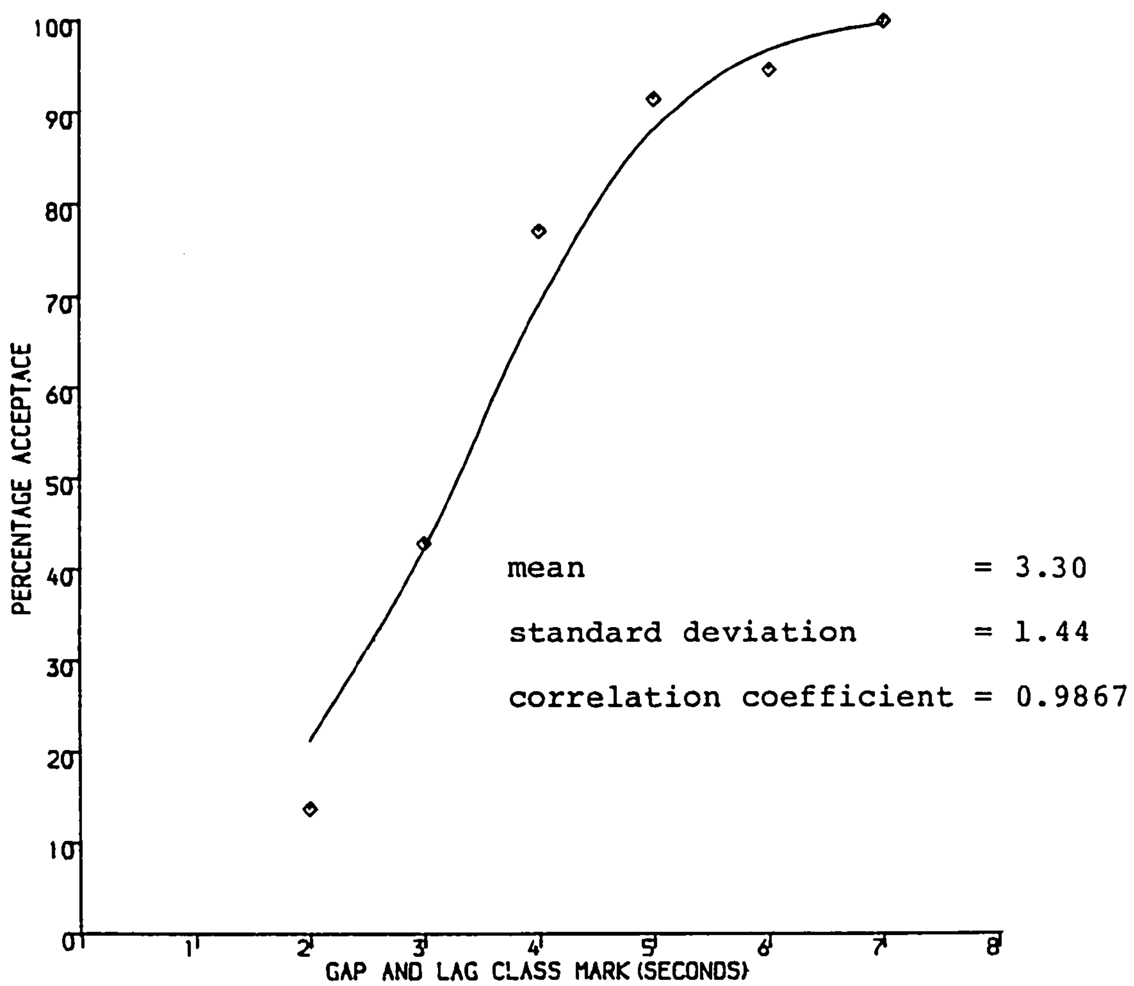


Figure 3.7.1-1 Lag and gap acceptance distribution for entry flow (passenger cars), M606 Motorway.

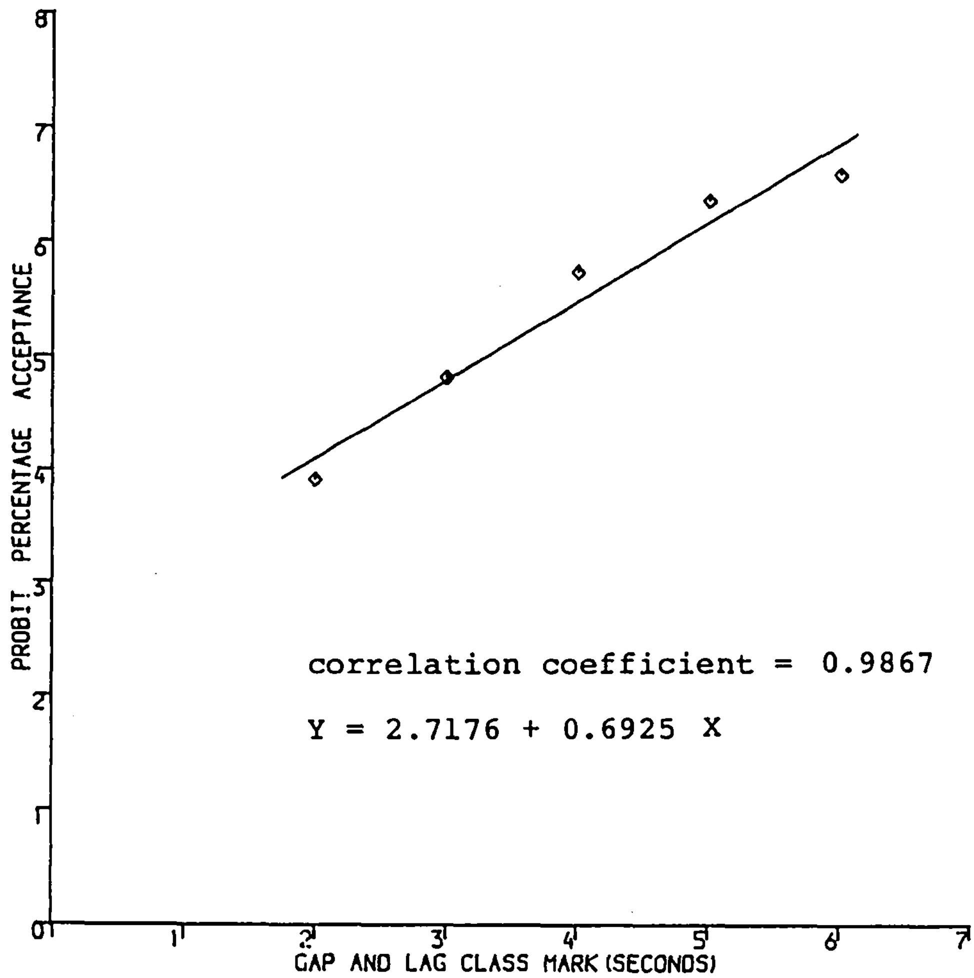


Figure 3.7.1-2 Probit of lag and gap acceptance for entry flow (passenger cars), M606 Motorway.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	-	3	0	-	-
1.5 ---- 2.49	-	15	0	-	-
2.5 ---- 3.49	5	12	29.41	4.46	28.10
3.5 ---- 4.49	6	4	60.00	5.25	63.10
4.5 ---- 5.49	9	1	90.00	6.28	89.40
5.5+	19	-	100.00	-	

Table 3.7.1-2 Classification of data and distribution fitting of lag and gap acceptance of entry movement (light goods vehicle),M606 Motorway.

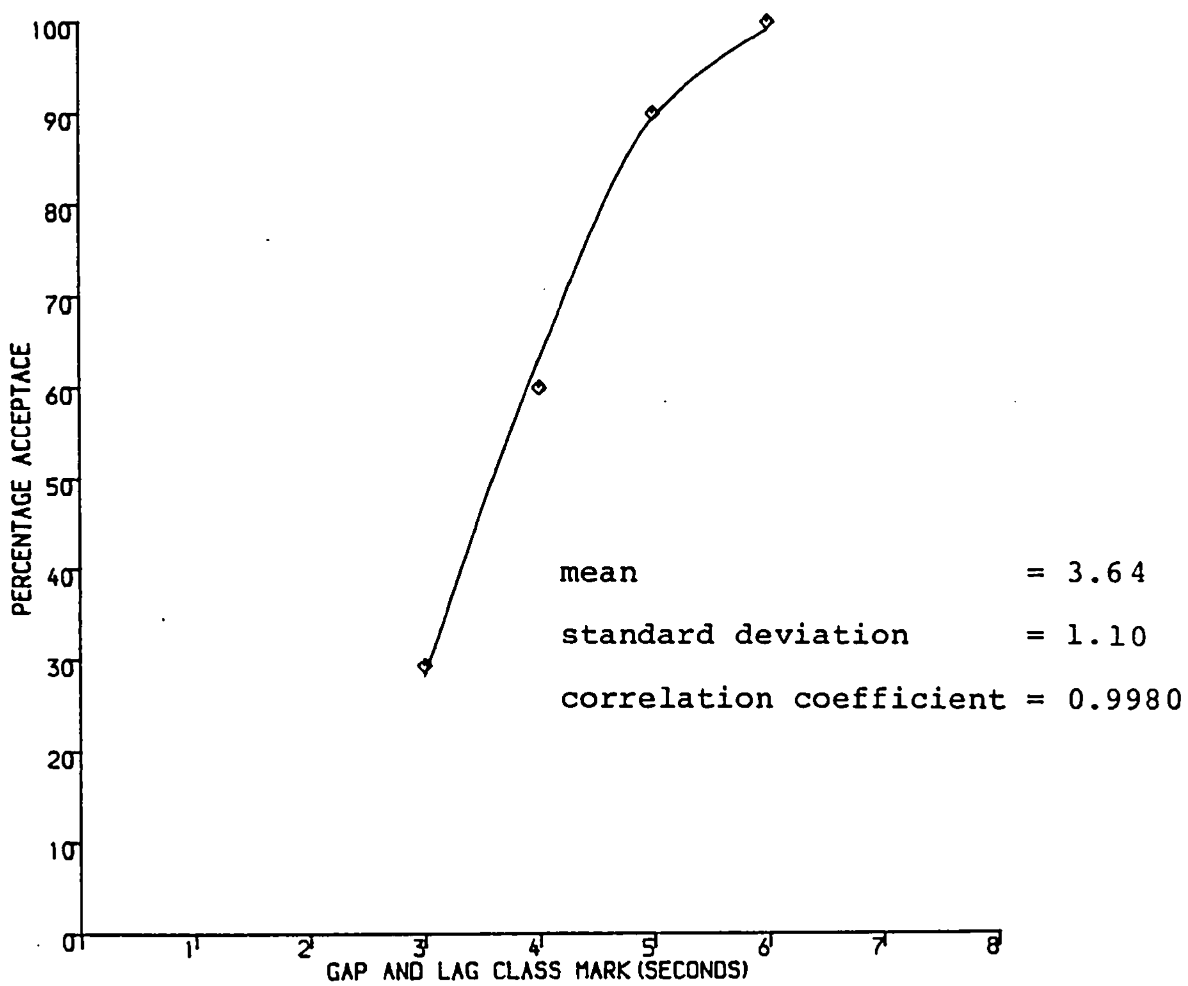


Figure 3.7.1-3 Lag and gap acceptance distribution for entry flow (light goods vehicle), M606 Motorway.

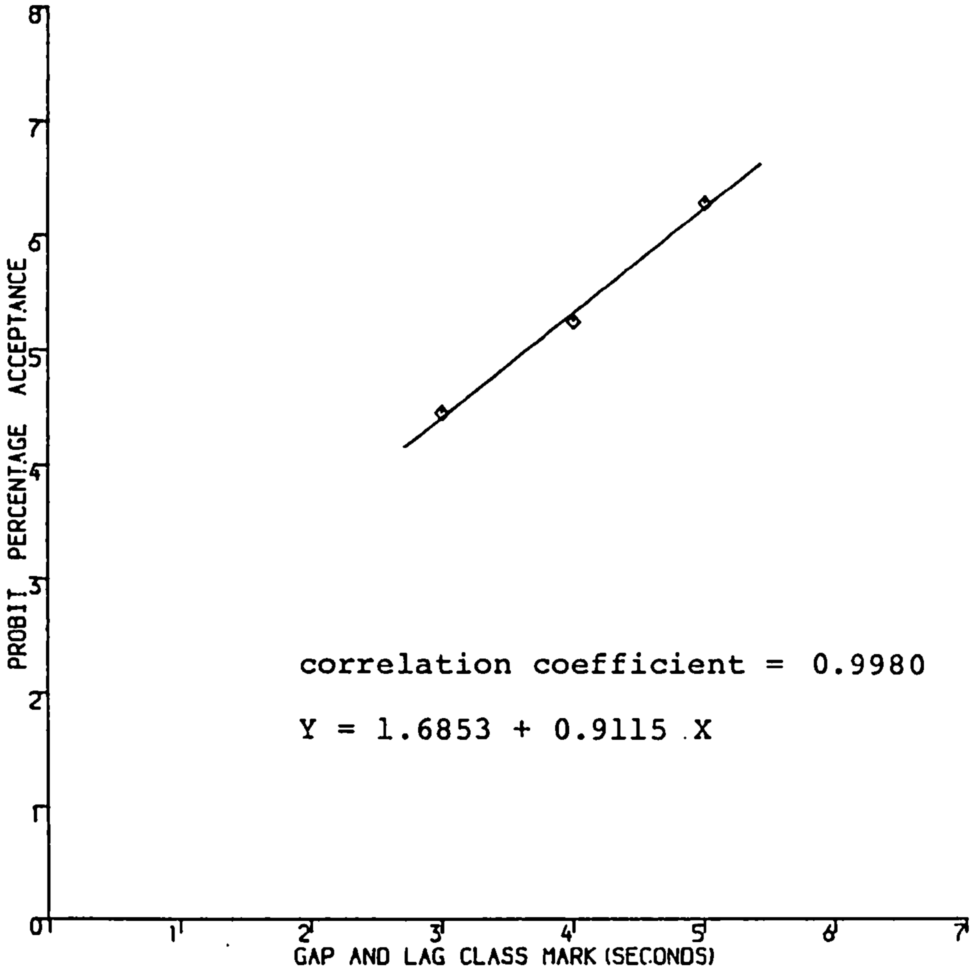


Figure 3.7.1-4 Probit of lag and gap acceptance for entry flow (light goods vehicle), M606 Motorway.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	0	4	-	-	-
1.5 ---- 2.49	0	23	-	-	-
2.5 ---- 3.49	4	14	22.22	4.24	29.12
3.5 ---- 4.49	7	4	37.00	4.67	54.00
4.5 ---- 5.49	15	2	88.24	6.19	77.34
5.5 ---- 6.49	31	0	100	-	97.72
6.5 ---- 7.49					
7.5 +					

Figure 3.7.1-3 Classification of data and distribution fitting of lag and gap acceptance of entry movement (heavy goods vehicle), M606 Motorway.

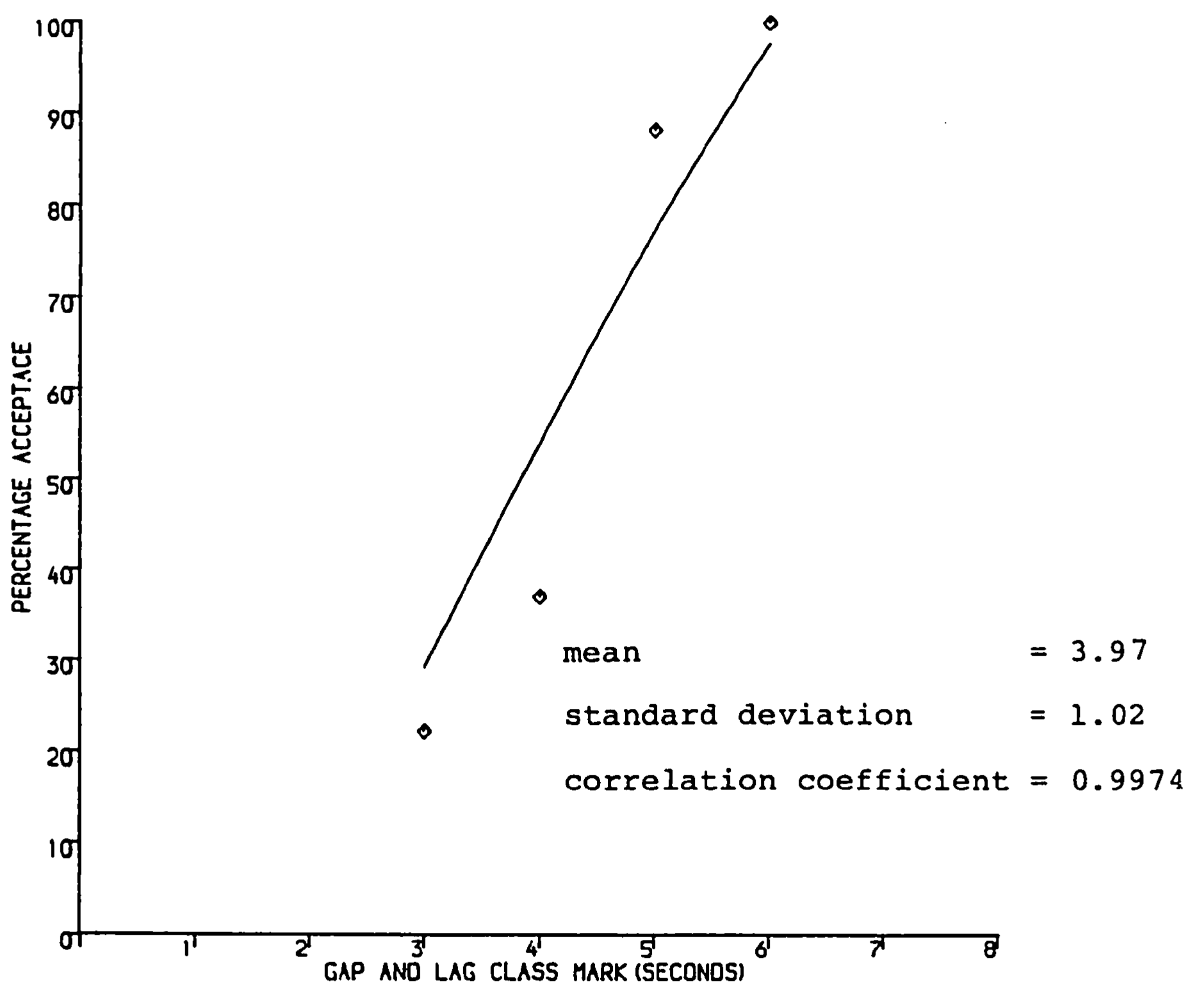


Figure 3.7.1-5 Lag and gap acceptance distribution for entry flow (heavy goods vehicle), M606 Motorway.

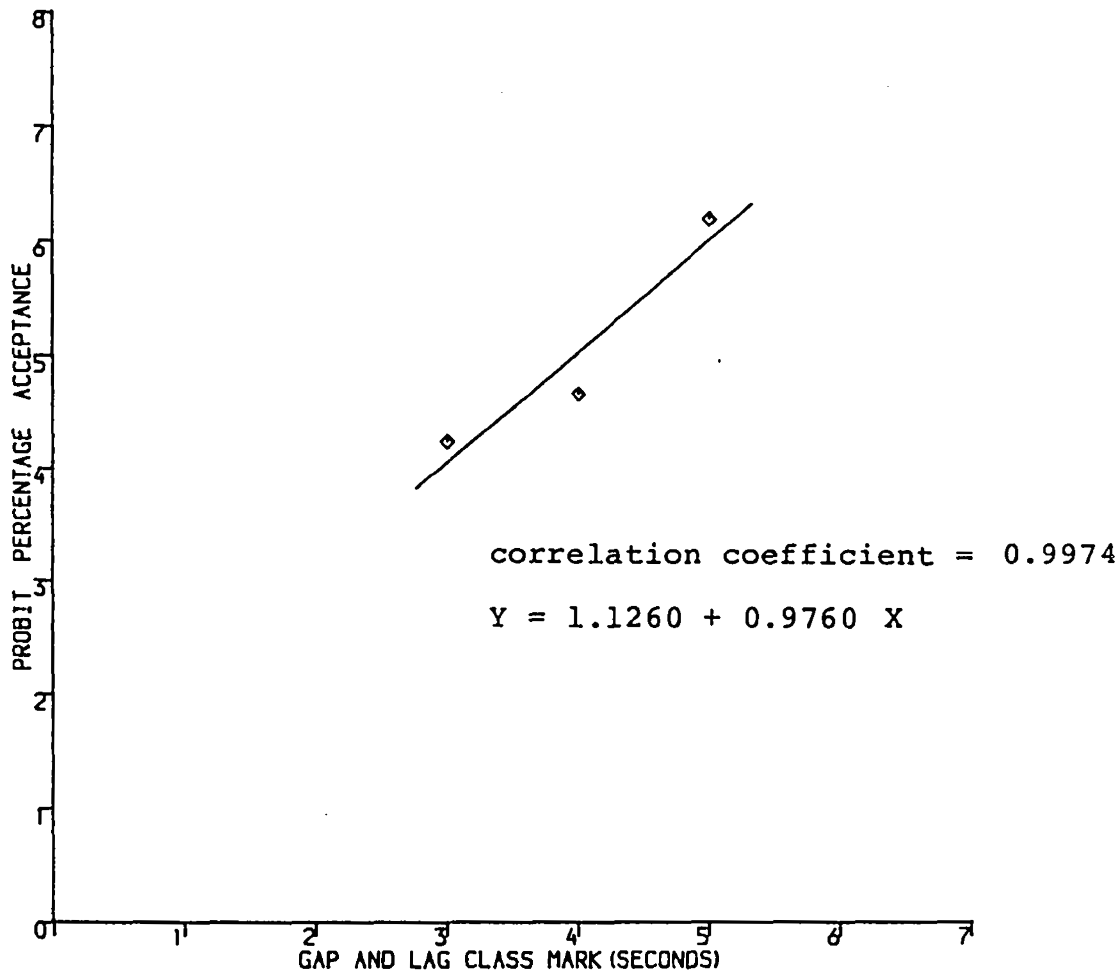


Figure 3.7.1-6 Probit of lag and gap acceptance for entry flow (heavy goods vehicle), M606 Motorway.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	0	2	0	-	-
1.5 ---- 2.49	0	8	0	-	-
2.5 ---- 3.49	2	12	14.30	3.93	13.90
3.5 ---- 4.49	3	4	42.90	4.82	44.70
4.5 ---- 5.49	4	1	80.00	5.84	79.40
5.5+	20	-	100	-	

Table 3.7.1-4 Classification of data and distribution fitting of lag and gap acceptance of entry movement (articulated goods vehicle),M606 Motorway.

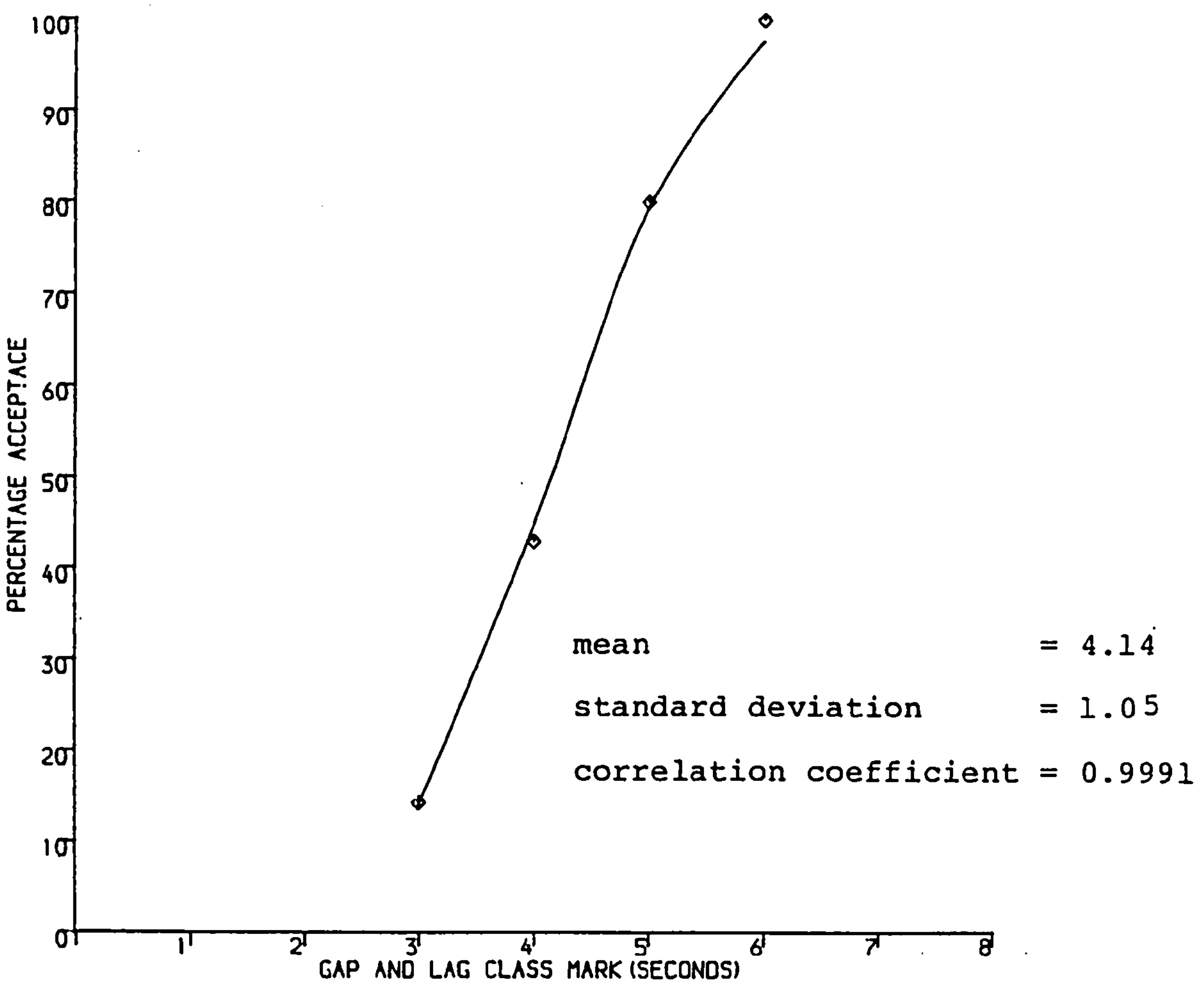


Figure 3.7.1-7 Lag and gap acceptance distribution for entry flow (articulated goods vehicle), M606 Motorway.

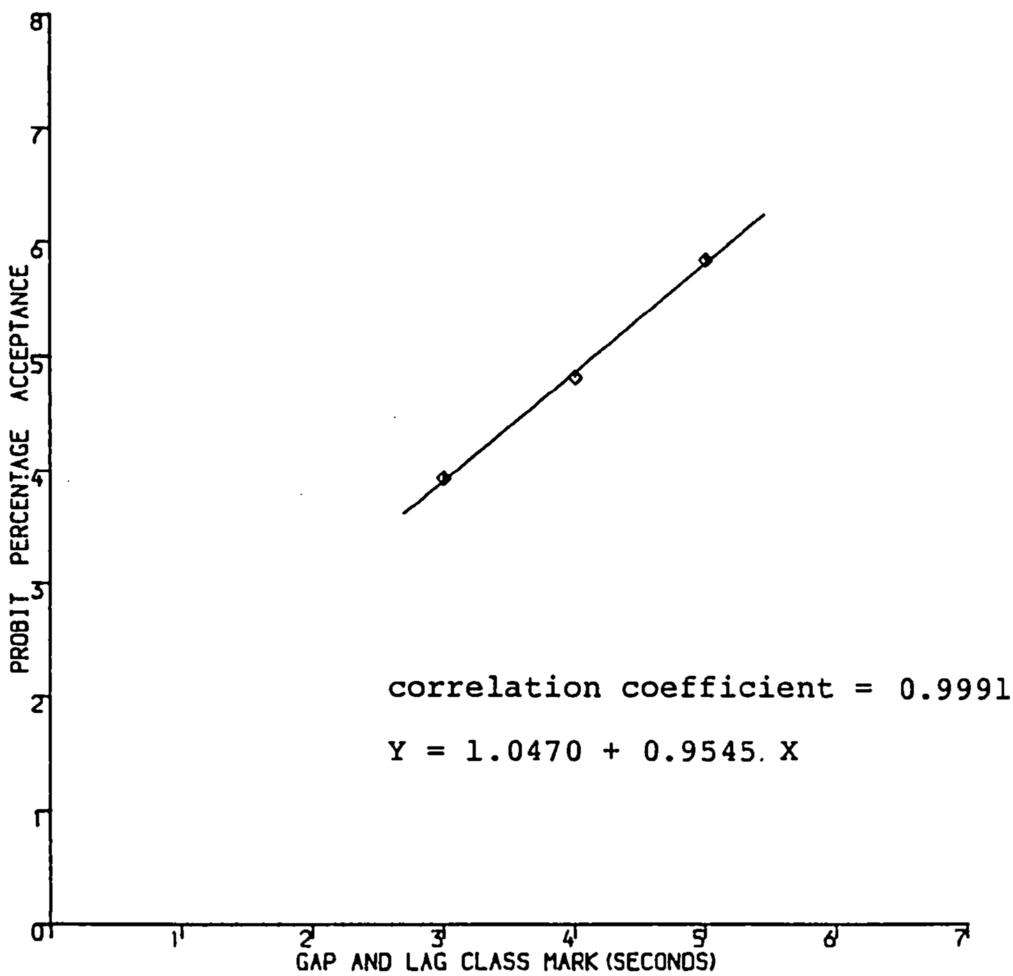


Figure 3.7.1-8 Probit of lag and gap acceptance for entry flow (articulated goods vehicle), M606 Motorway.

No. of table	Vehicle type	Regression Equation	Mean acceptance	Standard deviation	R
3.7.1-1	Cars	$Y = 2.17176 + 0.6925\ X$	3.30	1.44	0.9867
3.7.1-2	LGV	$Y = 1.6853 + 0.9115\ X$	3.64	1.10	0.9980
3.7.1-3	HGV	$Y = 1.1260 + 0.9760\ X$	3.97	1.03	0.9974
3.7.1-4	AGV	$Y = 1.0470 + 0.9545\ X$	4.14	1.05	0.9991

Table 3.7.1-5 Summary of results for gap acceptance studies at site (3) - (M606 Motorway).

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	0	47	-	-	-
1.5 ---- 2.49	16	72	18.20	4.09	15.93
2.5 ---- 3.49	28	39	42.00	4.80	46.81
3.5 ---- 4.49	64	16	80.00	5.84	80.23
4.5 ---- 5.49	89	3	97.00	6.88	96.40
5.5 ---- 6.49	233	0	100		
6.5 ---- 7.49					
7.5 +					

Table 3.7.1-6 Classification of data and distribution fitting of lag and gap acceptance of entry movement (passenger cars) ,Old Street.

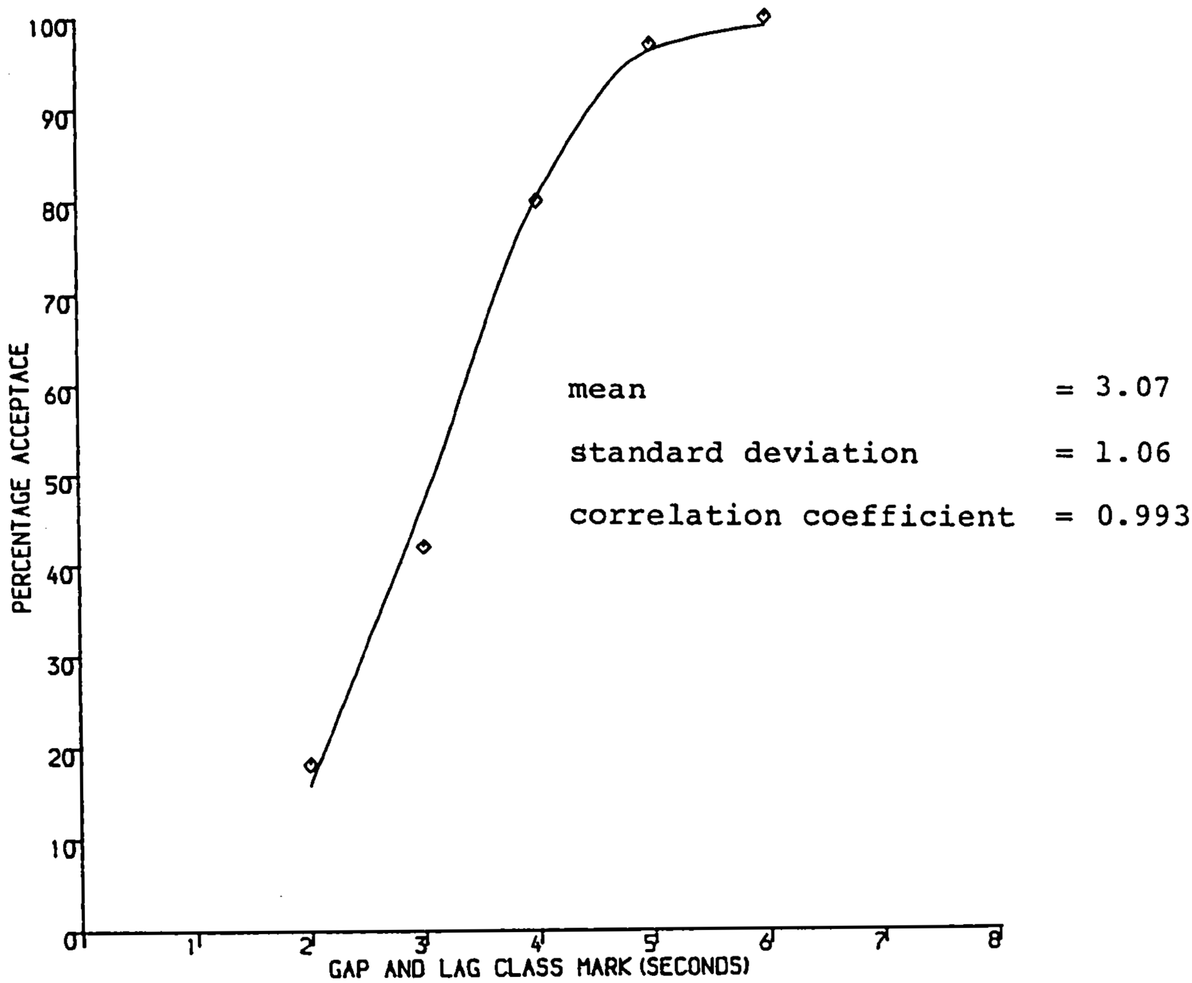
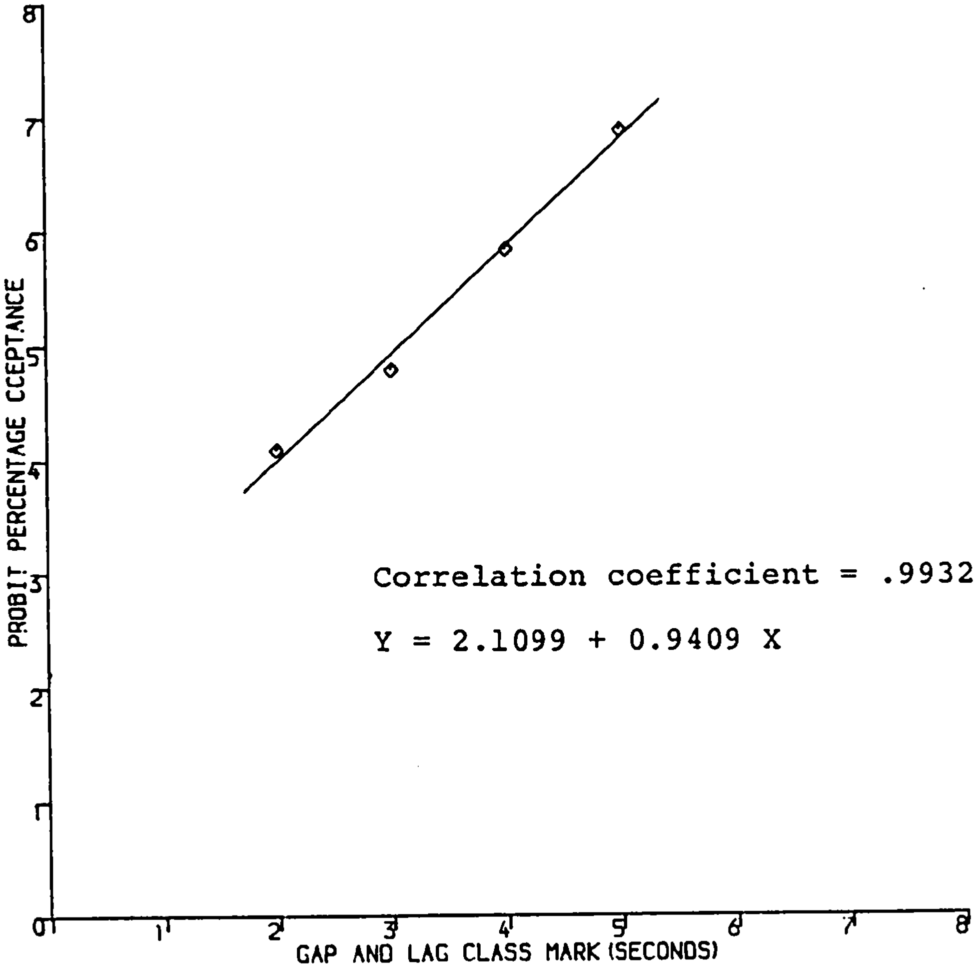


Figure 3.7.1-9 Lag and gap acceptance distribution for entry flow (passenger cars), Old Street.



Correlation coefficient = .9932

Y = 2.1099 + 0.9409 X

Figure 3.7.1-10 Probit of lag and gap acceptance for entry flow (passenger cars), Old Street.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	0	55	-	-	
1.5 ---- 2.49	8	63	11.30	3.79	8.85
2.5 ---- 3.49	16	46	26.00	4.36	29.12
3.5 ---- 4.49	30	21	59.00	5.23	57.93
4.5 ---- 5.49	58	10	85.00	6.04	82.90
5.5 ---- 6.49	62	3	95.00	6.65	95.70
6.5 ---- 7.49	159	0	100		
7.5 +					

Table 3.7.1- 7 Classification of data and distribution fitting of lag and gap acceptance of entry movement (light goods vehicle), Old Street.

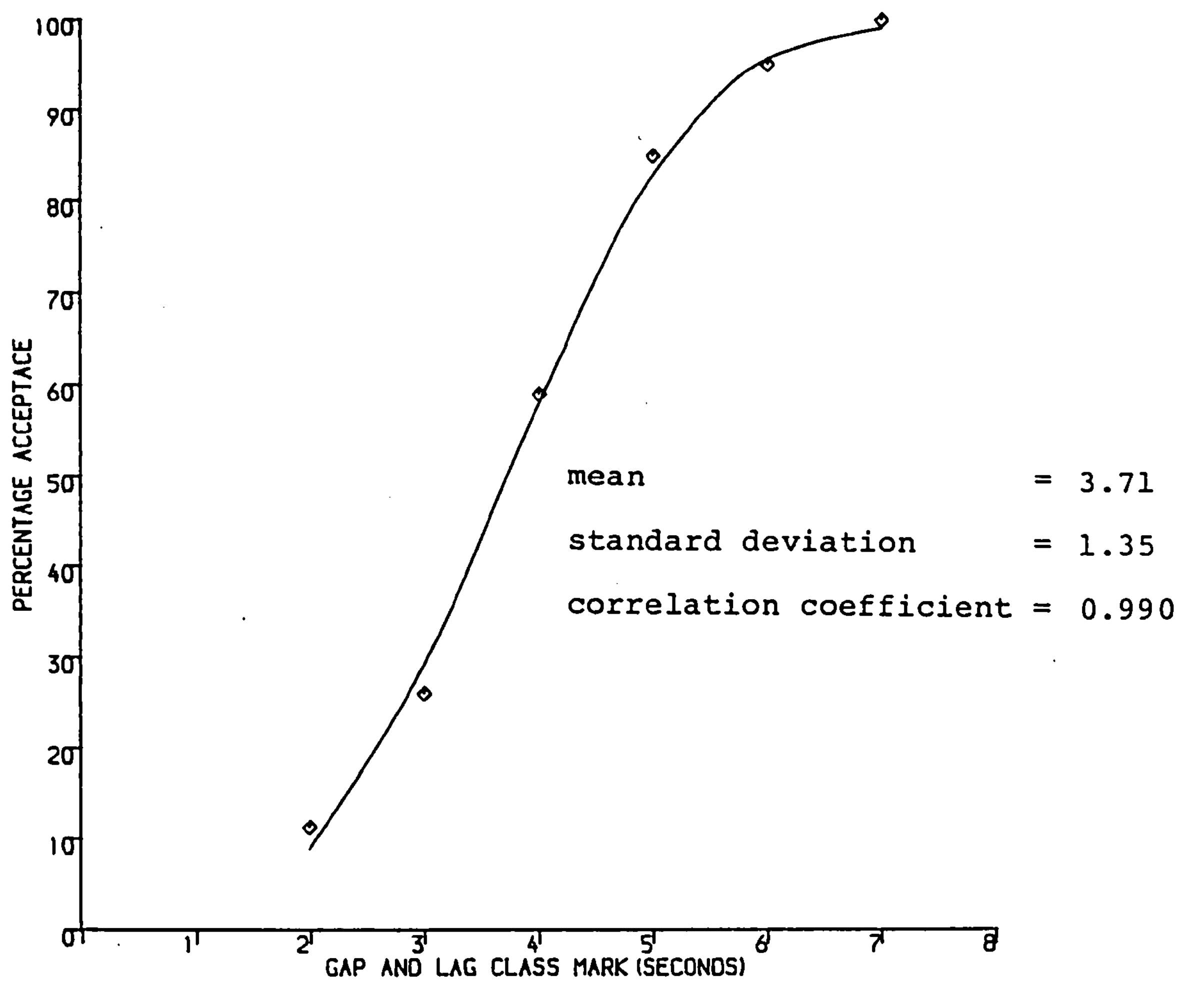


Figure 3.7.1-11 Lag and gap acceptance distribution for entry flow (light goods vehicles), Old Street.

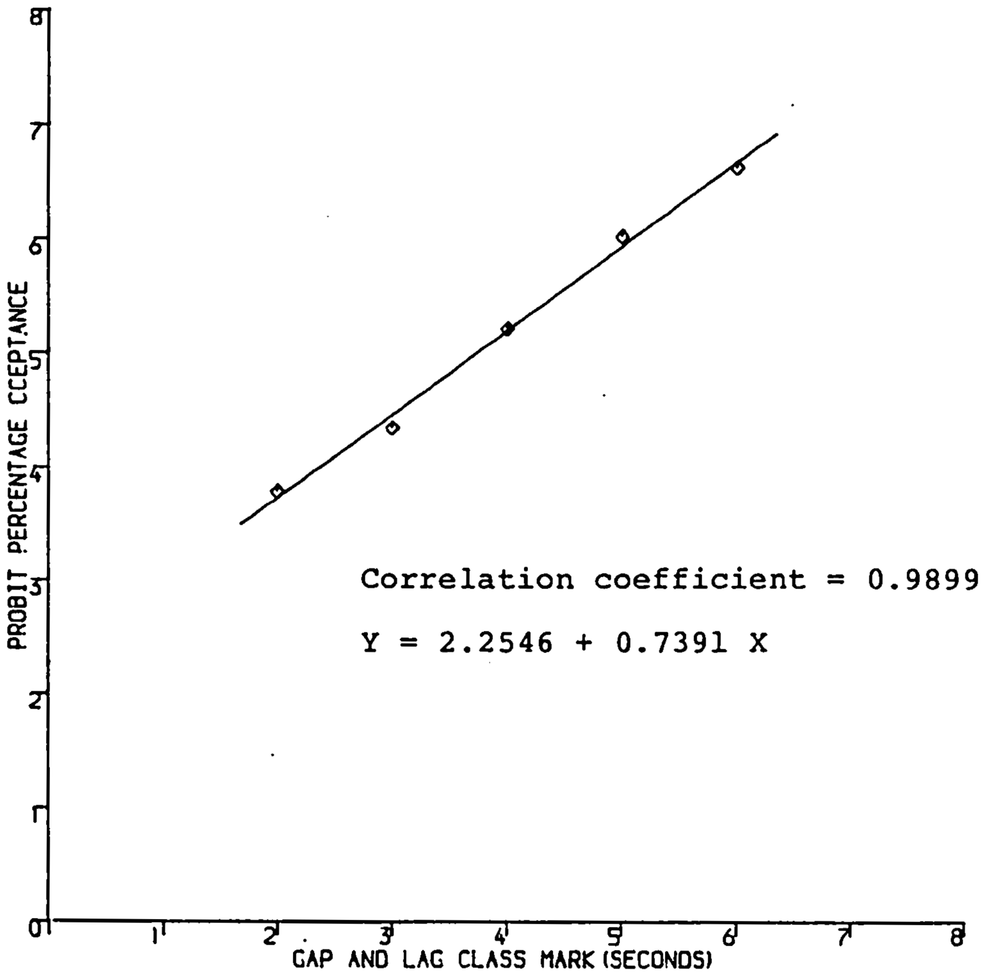


Figure 3.7.1-12 Probit of lag and gap acceptance for entry flow (light goods vehicles), Old Street.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	0	41	--	-	-
1.5 ---- 2.49	0	53	-	-	-
2.5 ---- 3.49	3	66	4.40	3.29	4.00
3.5 ---- 4.49	7	51	12.00	3.83	17.10
4.5 ---- 5.49	15	25	38.00	4.70	46.01
5.5 ---- 6.49	28	10	74.00	5.64	77.34
6.5 ---- 7.49	39	2	95.00	6.65	94.30
7.5 +	213	0	100	-	-

Table 3.7.1-8 Classification of data and distribution fitting of lag and gap acceptance of entry movement (heavy goods vehicle), Old Street.

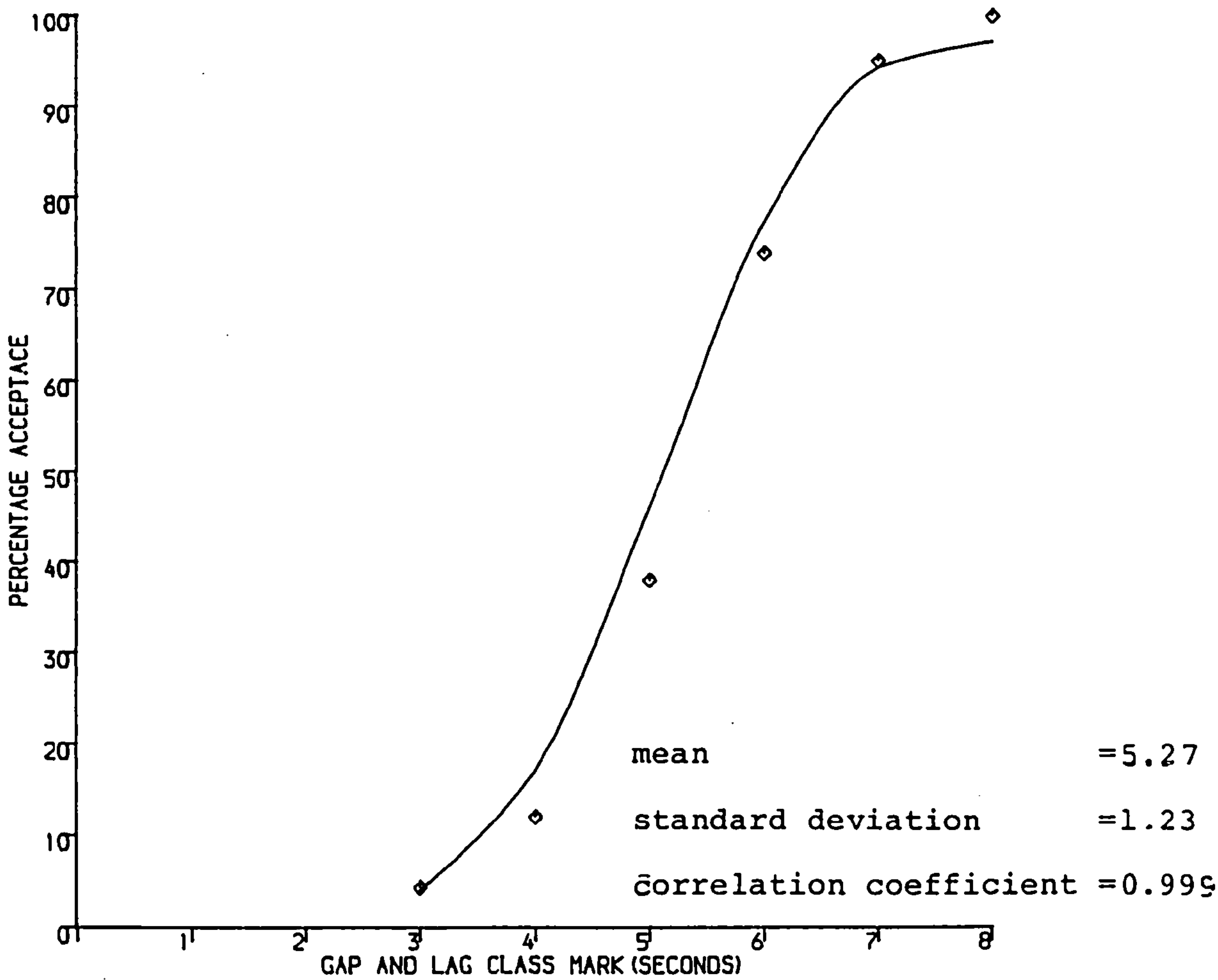


Figure 3.7.1-13 Lag and gap acceptance distribution for entry flow (heavy goods vehicles), Old Street.

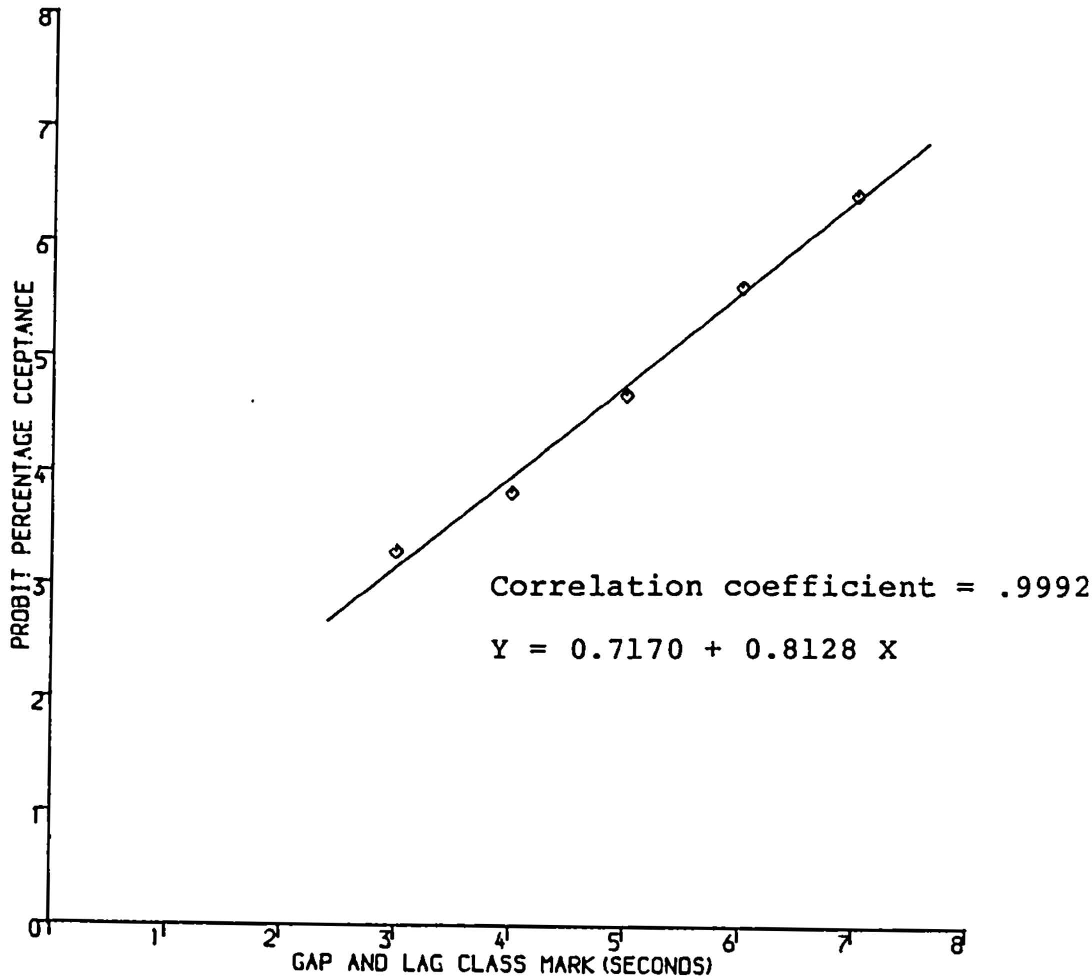


Figure 3.7.1-14 Probit of lag and gap acceptance for entry flow (heavy goods vehicles), Old Street.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	0	19	-	-	-
1.5 ---- 2.49	0	32	-	-	-
2.5 ---- 3.49	0	37	-	-	-
3.5 ---- 4.49	6	48	11.00	3.77	9.18
4.5 ---- 5.49	18	56	24.00	4.19	27.43
5.5 ---- 6.49	37	31	54.00	5.10	57.93
6.5 ---- 7.49	69	12	85.00	6.04	82.40
7.5 ---- 8.49	77	4	95.00	6.65	95.70
8.5+	84	0	100	-	-

Table 3.7.1-9 Classification of data and distribution fitting of lag and gap acceptance of entry movement (articulated goods vehicle), Old Street.

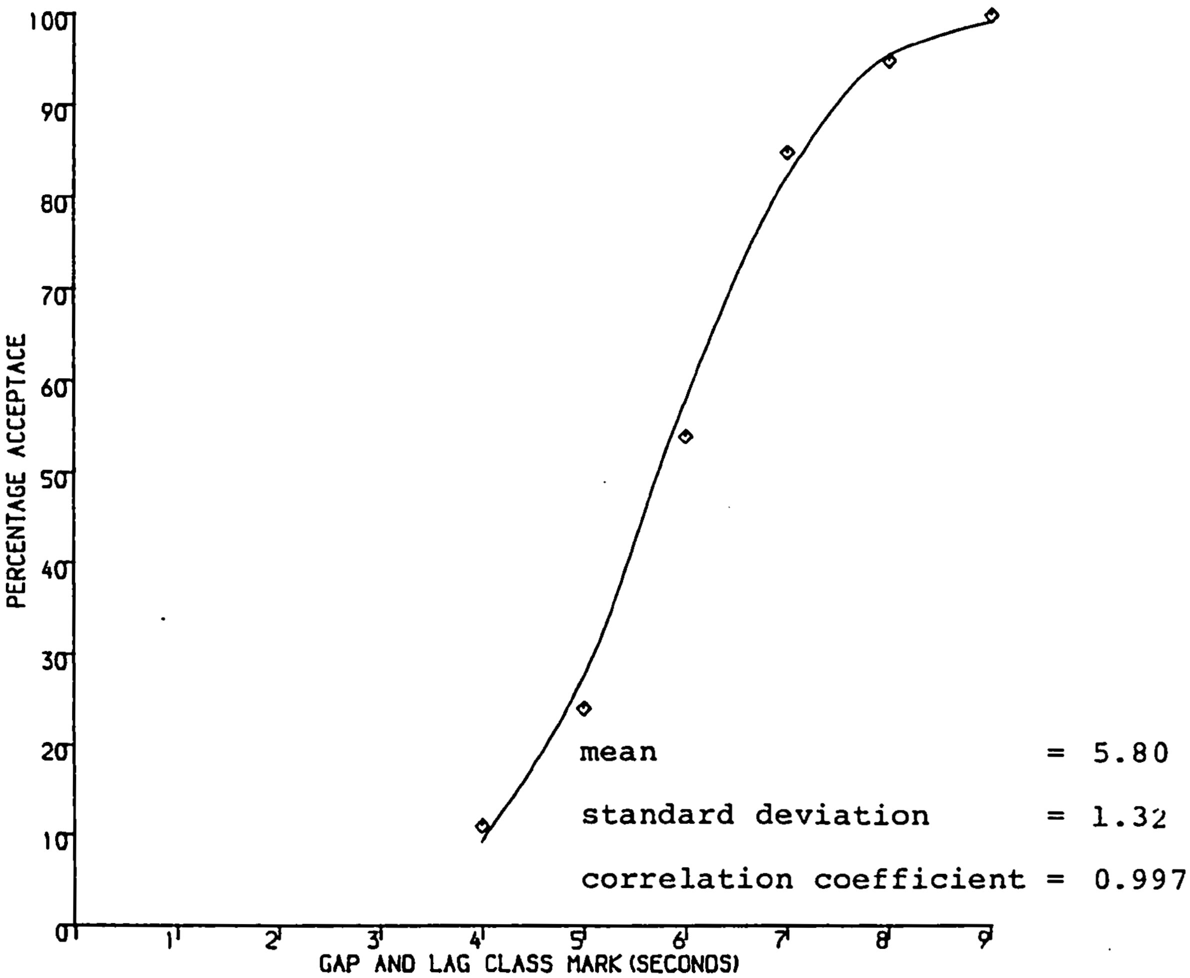


Figure 3.7.1-15 Lag and gap acceptance distribution for entry flow (articulated goods vehicles), Old Street.

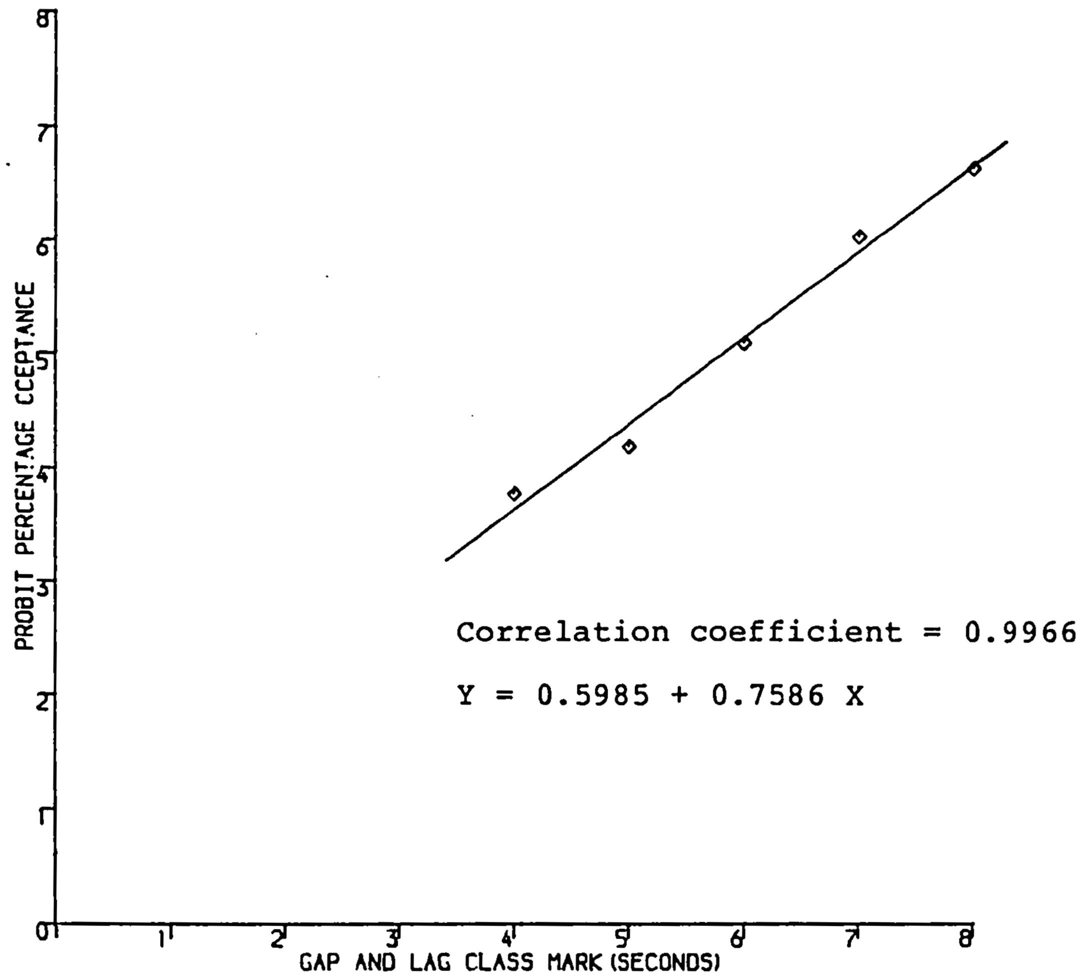


Figure 3.7.1-16 Probit of lag and gap acceptance for entry flow (articulated goods vehicles), Old Street.

No. of table	Vehicle type	Regression Equation	Mean acceptance	Standard deviation	R
3.7.1-6	Cars	Y = 2.1099 + 0.9409 X	2.24	1.06	0.993
3.7.1-7	LGV	Y = 2.2546 + 0.7391 X	3.72	1.35	0.990
3.7.1-8	HGV	Y = 0.7170 + 0.8128 X	5.27	1.23	0.999
3.7.1-9	AGV	Y = 0.5985 + 0.7586 X	5.80	1.32	0.997

Table 3.7.1-10 Summary of results for gap acceptance studies at Site 1 (Old Street - London).

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	-	58	-		
1.5 ---- 2.49	20	76	20.83	4.19	19.77
2.5 ---- 3.49	37	45	45.12	4.88	44.03
3.5 ---- 4.49	85	21	80.20	5.85	70.88
4.5 ---- 5.49	89	11	89.00	6.23	89.43
5.5 ---- 6.49	66	3	95.65	6.72	97.32
6.5 ---- 7.49	58	-	100	-	
7.5 +					

Table 3.7.1-11 Classification of data and distribution fitting of lag and gap acceptance of entry movement (cars only), City Road.

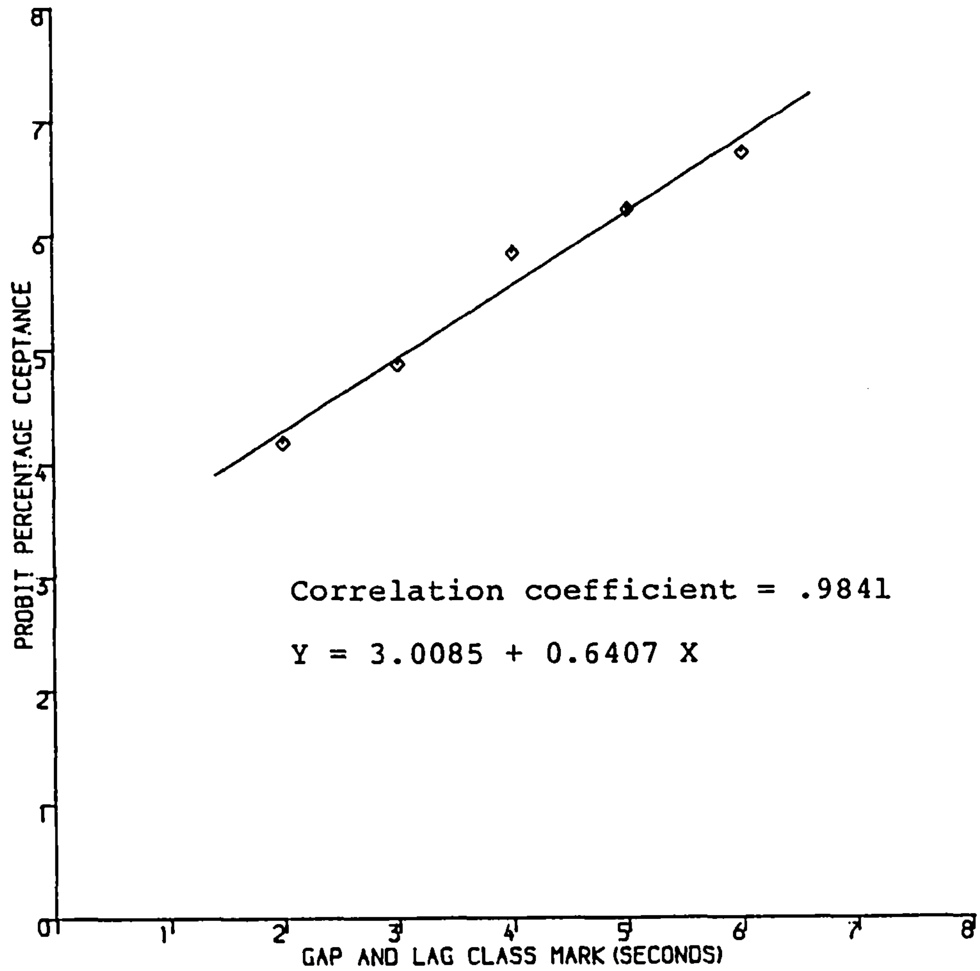


Figure 3.7.1-18 Probit of lag and gap acceptance for entry flow (passenger cars), City Road.

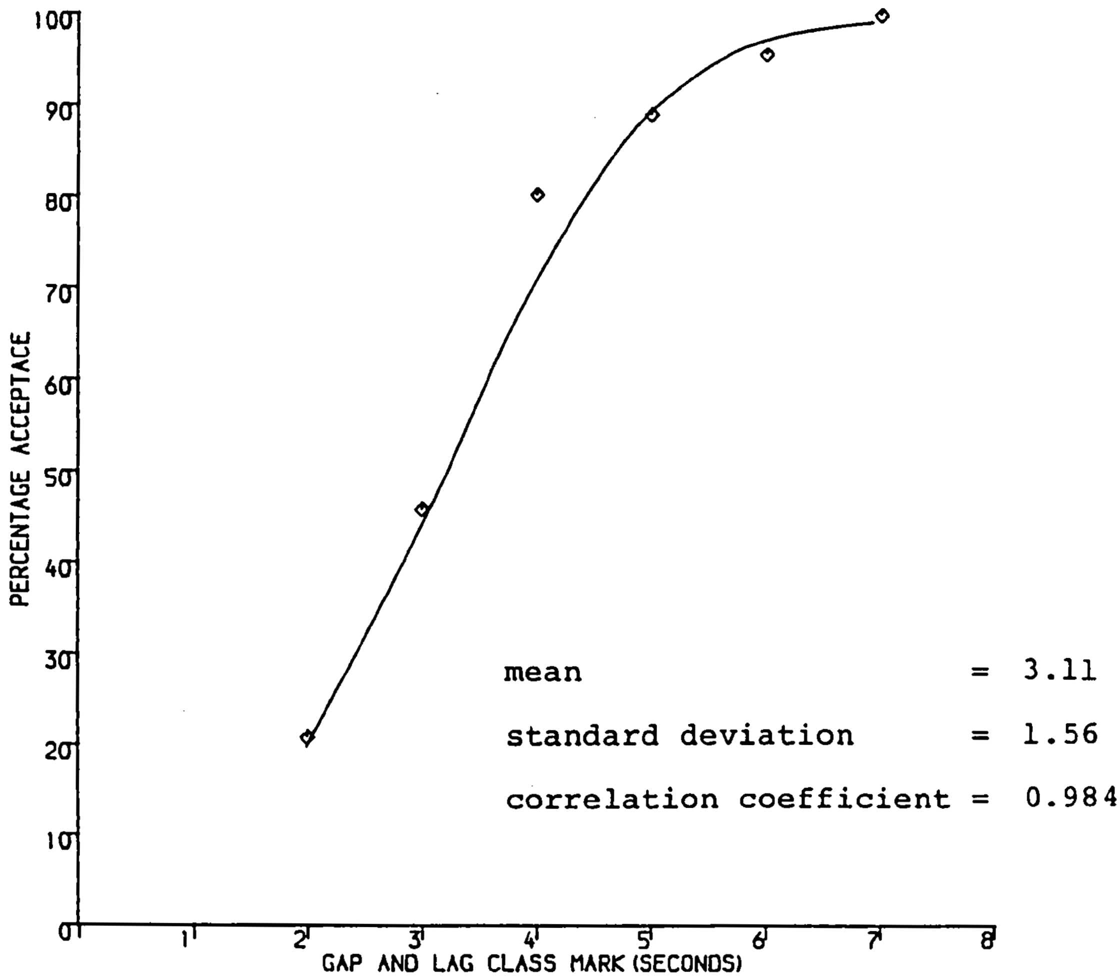


Figure 3.7.1-17 Lag and gap acceptance distribution for entry flow (passenger cars), City Road.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	-	37	-	-	-
1.5 ---- 2.49	1	45	2.17	2.99	2.12
2.5 ---- 3.49	6	59	9.23	3.67	8.69
3.5 ---- 4.49	13	41	24.07	4.30	23.58
4.5 ---- 5.49	27	30	47.37	4.94	47.19
5.5 ---- 6.49	33	17	66.00	5.41	72.57
6.5 ---- 7.49	47	4	92.16	6.42	89.58
7.5 +	62	-	100	-	-

Table 3.7.1-12 Classification of data and distribution fitting of lag and gap acceptance of entry movement (light goods vehicles), City Road.

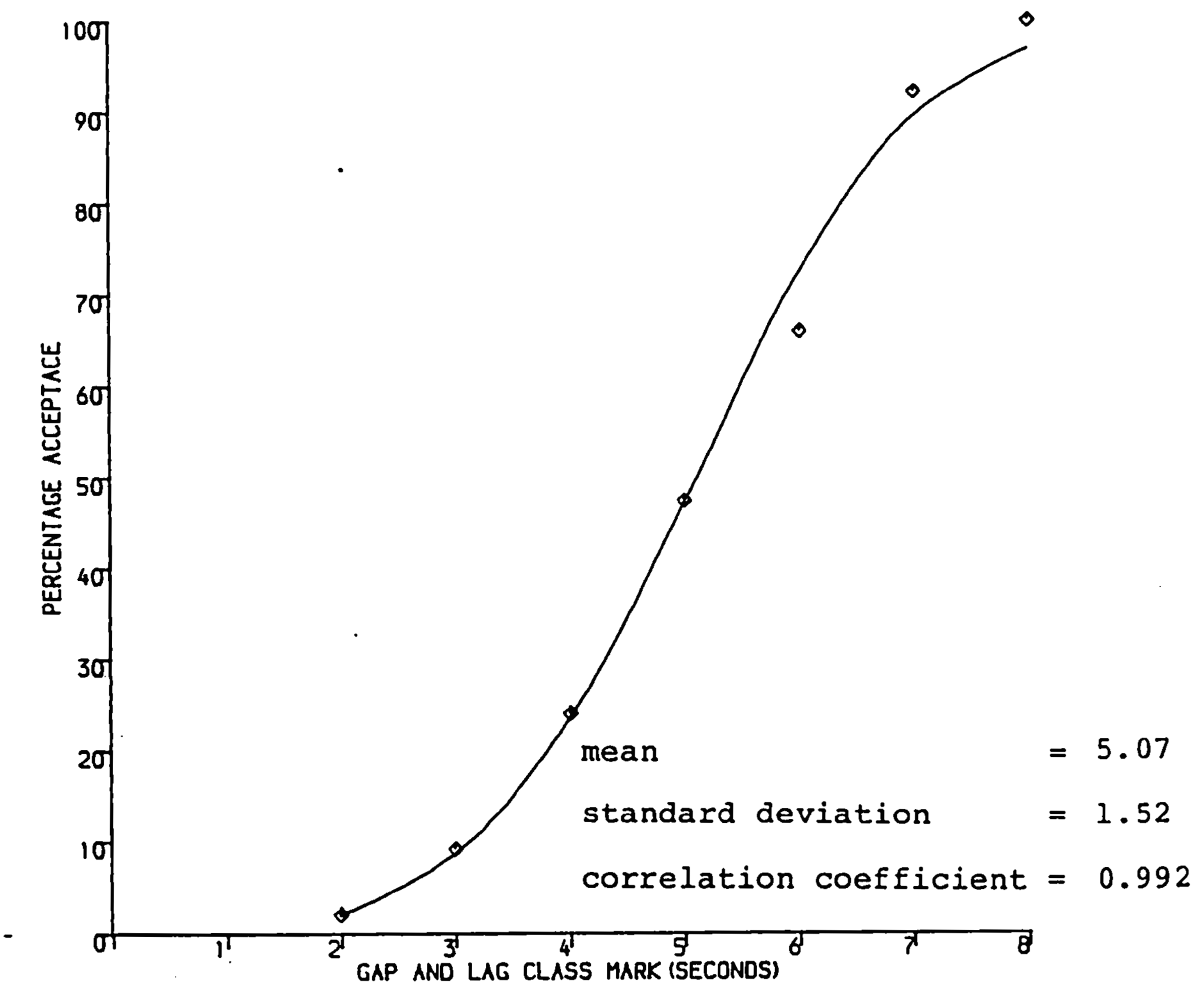


Figure 3.7.1-19 Lag and gap acceptance distribution for entry flow (light goods vehicles), City Road.

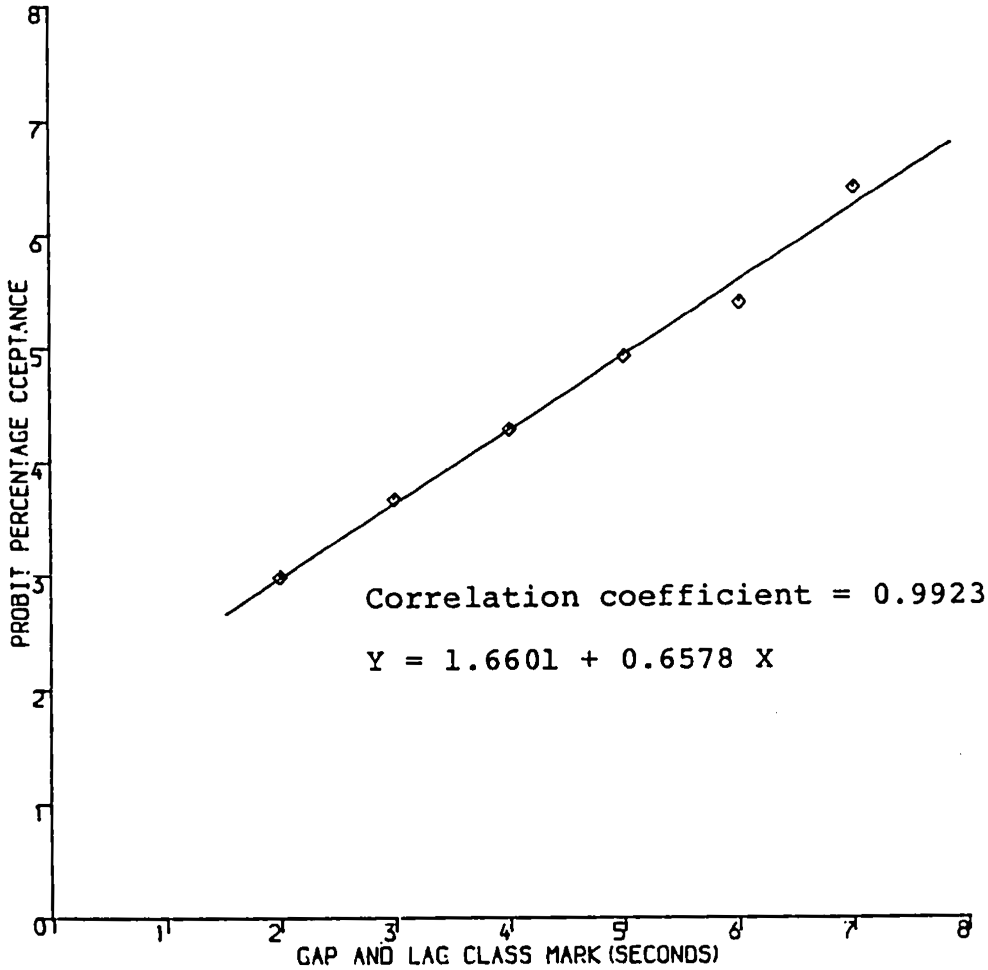


Figure 3.7.1-20 Probit of lag and gap acceptance for entry flow (light goods vehicles), City Road.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	-	23	-	-	-
1.5 ---- 2.49	-	37	-	-	-
2.5 ---- 3.49	5	43	10.42	3.74	6.68
3.5 ---- 4.49	15	57	20.83	4.19	19.08
4.5 ---- 5.49	22	41	34.92	4.61	34.46
5.5 ---- 6.49	29	26	52.73	5.07	65.44
6.5 ---- 7.49	42	11	79.25	5.82	84.13
7.5 ---- 8.49	58	2	96.67	6.84	95.35
8.5+	29	-	100	-	-

Table 3.7.1-13 Classification of data and distribution fitting of lag and gap acceptance of entry movement (heavy goods vehicles), City Road.

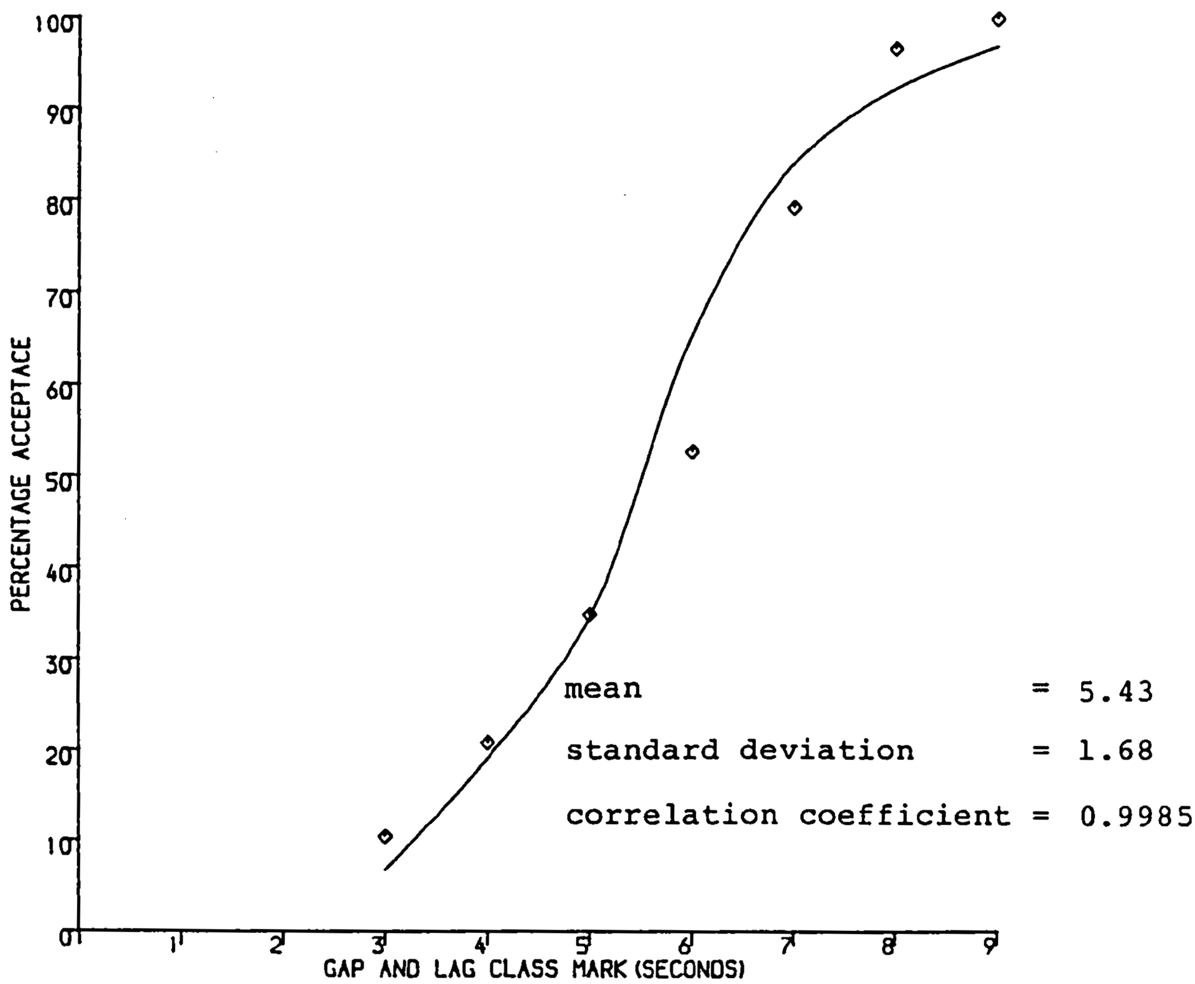


Figure 3.7.1-21 Lag and gap acceptance distribution for entry flow (heavy goods vehicles), City Road.

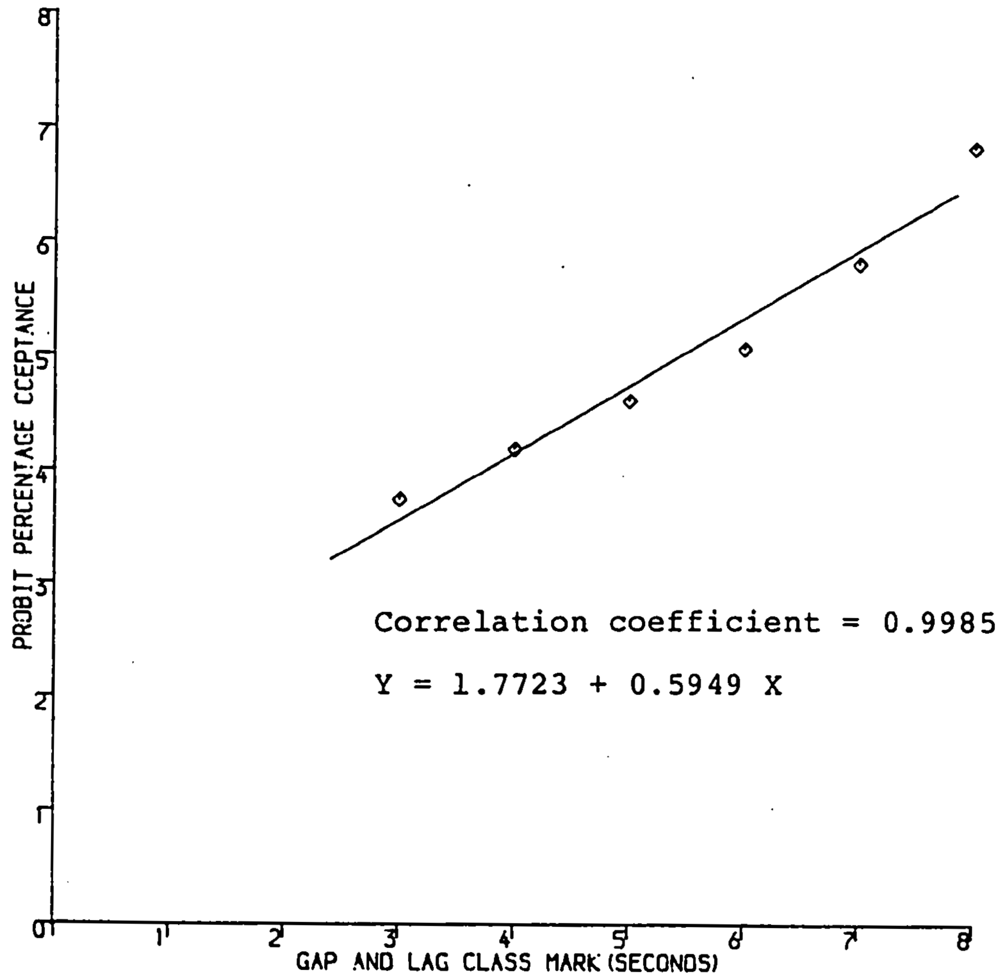


Figure 3.7.1-22 Probit of lag and gap acceptance for entry flow (heavy goods vehicles), City Road.

Lag and gap class mark "t" seconds	Number accepted in class	Number rejected in class	Percentage of acceptance	Probit	Theoretical % acceptance cumulative normal distribution
0.5 ---- 1.49	-	23	-	-	-
1.5 ---- 2.49	-	37	-	-	-
2.5 ---- 3.49	5	43	10.42	3.74	6.68
3.5 ---- 4.49	15	57	20.83	4.19	19.08
4.5 ---- 5.49	22	41	34.92	4.61	34.46
5.5 ---- 6.49	29	26	52.73	5.07	65.44
6.5 ---- 7.49	42	11	79.25	5.82	84.13
7.5 ---- 8.49	58	2	96.67	6.84	95.35
8.5+	29	-	100	-	-

Table 3.7.1-14 Classification of data and distribution fitting of lag and gap acceptance of entry movement (articulated goods vehicles), City Road.

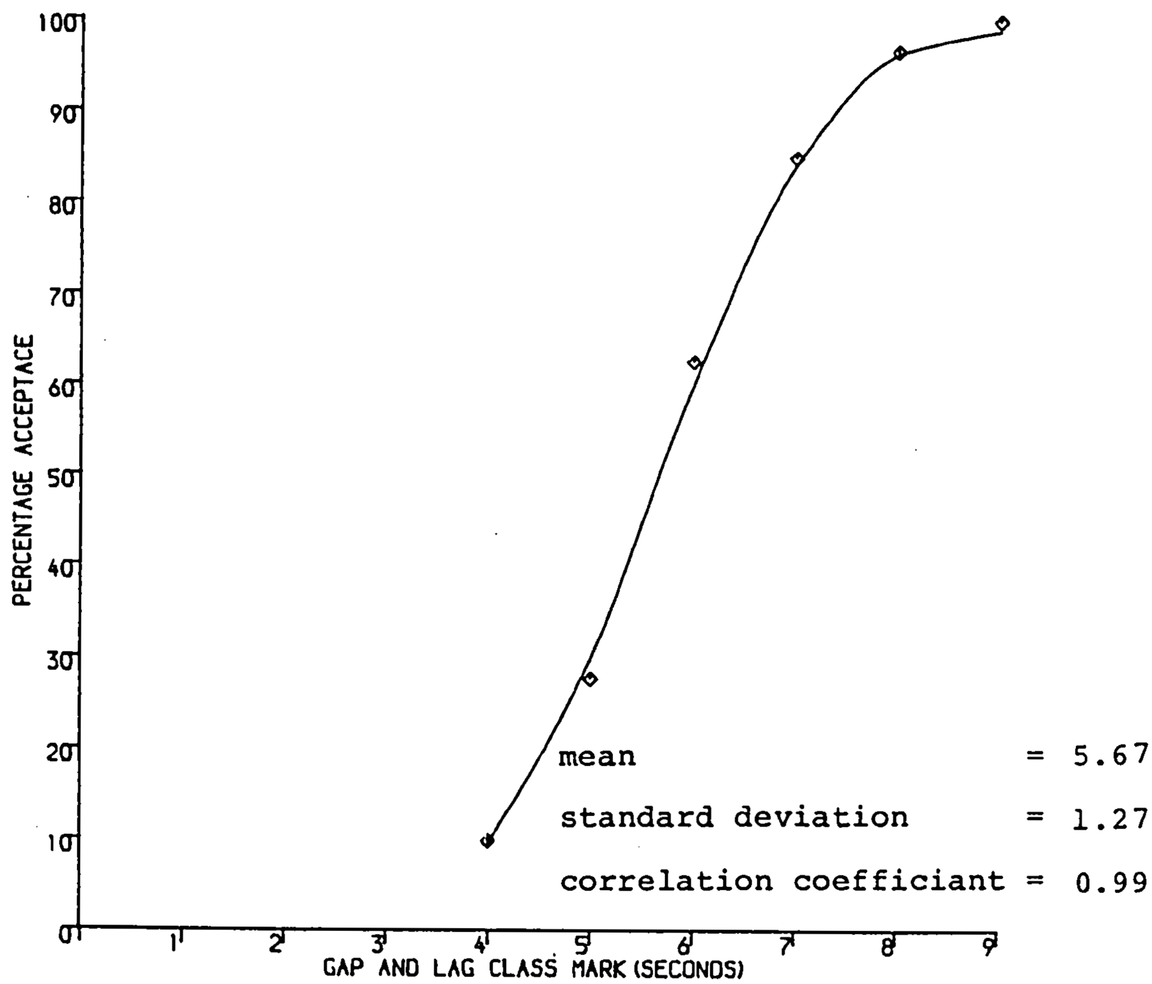


Figure 3.7.1-23 Lag and gap acceptance distribution for entry flow (articulated goods vehicles), City Road.

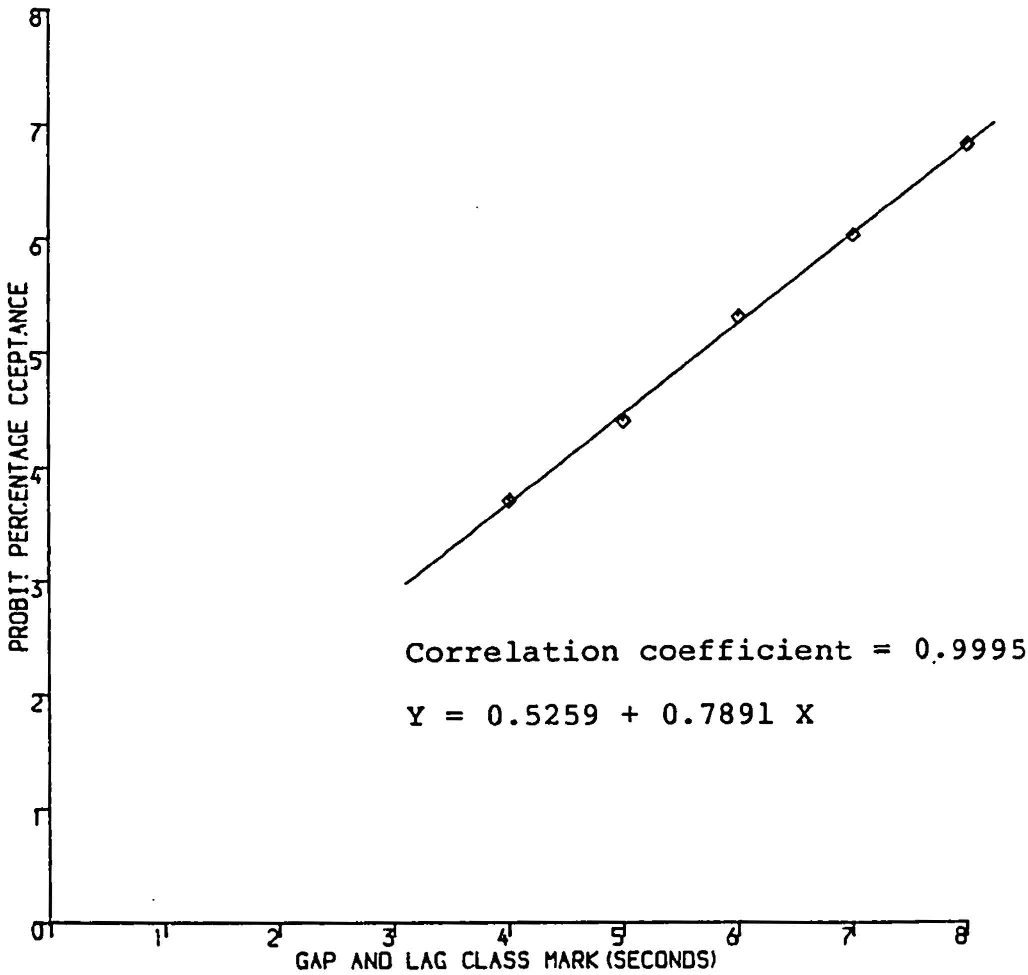


Figure 3.7.1-24 Probit of lag and gap acceptance for entry flow (articulated goods vehicles), City Road.

No. of table	Vehicle type	Regression Equation	Mean acceptance Sec.	Standard deviation	R
3.7.1-11	Cars	$Y = 3.0085 + 0.6407\ X$	3.11	1.56	0.984
3.7.1-12	LGV	$Y = 1.6601 + 0.6578\ X$	5.08	1.52	0.992
3.7.1-13	HGV	$Y = 1.7723 + 0.5949\ X$	5.43	1.68	0.9985
3.7.1-14	AGV	$Y = 0.5259 + 0.7891\ X$	5.67	1.27	0.999

Table 3.7.1-15 Summary of results for gap acceptance studies at Site 2 (City Road - London).

3.7.2 - <u>Gap accepted range and vehicle type relationship</u>

At the three sites observed, the results showed a wide range of variations in the accepted gap sizes corresponding to their vehicle type. For each vehicle type the minimum gap size accepted was noted and the maximum value was controlled by zero number of rejection.

Tables (3.7.2-1) to (3.7.2-3) show these variations and their corresponding number of observations for each type of vehicle considered. Figure (3.7.2-1) shows the comparison between the results of the three sites under study.

It was found that at sites (1) and (2) the minimum values of gaps accepted were the same for each type of vehicle, while for site (3) they differed slightly. The maximum gap sizes to be rejected were lower at site (3) than those of sites (1) and (2). The maximum for articulated goods vehicle for example was 8.5 seconds at site (1) while it was 6.5 seconds at site (3).

This shows again that the effect of vehicle type in the semi-rural situation is less than that in the urban situation.

Vehicle Type	Minimum gap observed to be accepted	Maximum gap observed to be accepted	Number of observations
Cars	2	5.5	430
LGV	2	6.5	333
HGV	3	7.5	305
AGV	4	8.5	291

Table 3.7.2-1 Vehicle type gap sizes for site (1) - Old Street, London.

Vehicle Type	Minimum gap observed to be accepted	Maximum gap observed to be accepted	Number of observations
Cars	2	6.5	355
LGV	2	7.5	189
HGV	3	8.5	200
AGV	4	8.9	200

Table 3.7.2-2 Vehicle type gap sizes for site (2) - City Road, London.

Vehicle Type	Minimum gap observed to be accepted	Maximum gap observed to be accepted	Number of observations
Cars	2	5.5	538
LGV	2	5.5	39
HGV	3	6.0	57
AGV	3	6.5	29

Table 3.7.2-3 Vehicle type gap sizes for site (3) - M606, Bradford.

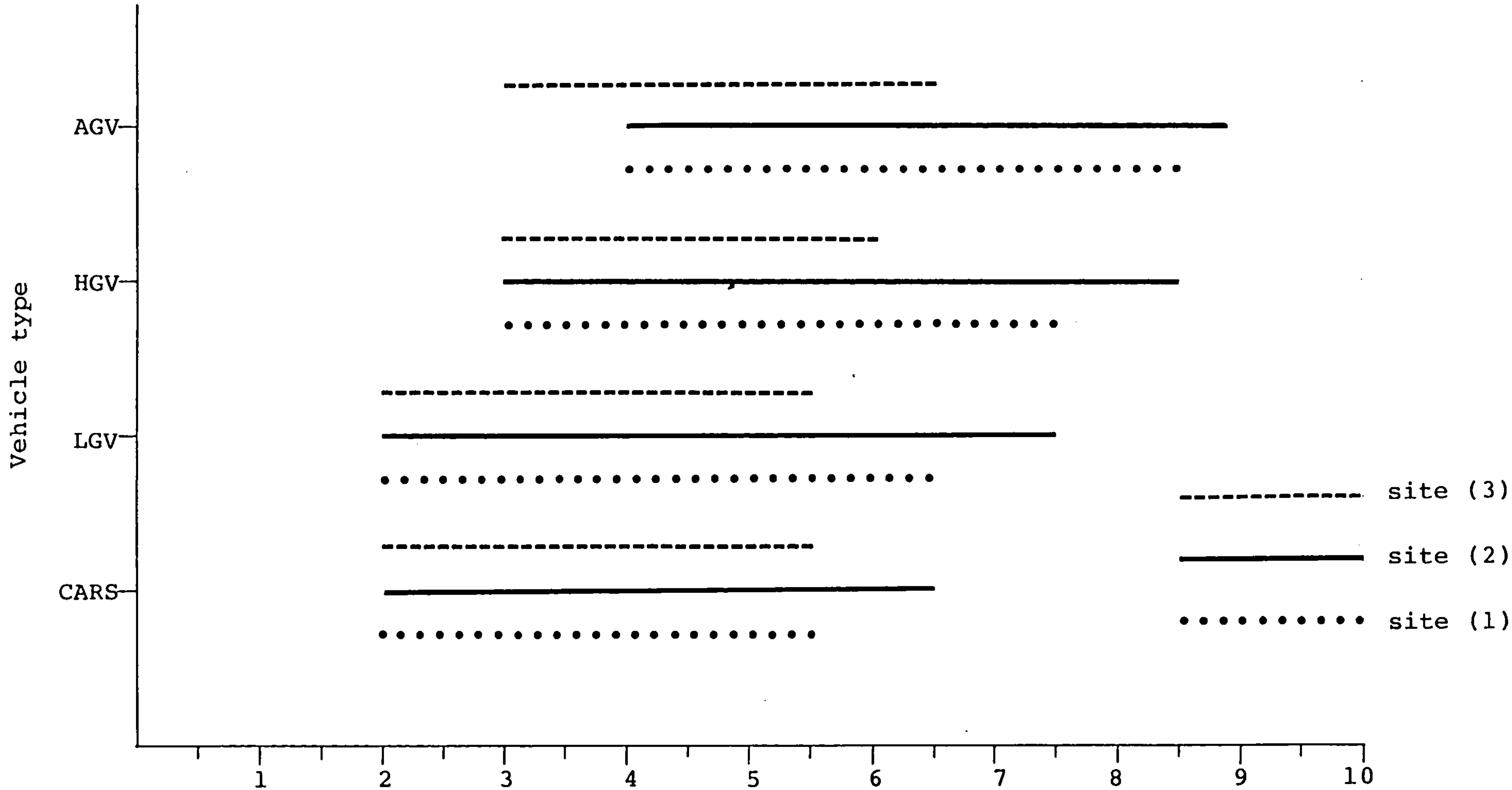


Figure 3.7.2-1 Gap accepted ranges for the three sites.

3.8 - Passenger car unit analysis

3.8.1 - Procedure adopted for computing passenger car units

The headway criteria was considered in the computation of passenger car units at roundabout entries. A number of saturated intervals were analysed to study the rate of discharge of vehicles. During each saturated interval the number of vehicles discharged was considered to be related to a constant factor. This factor is the average headway between passenger cars entering the circulating section.

The analysis was also based on the criterion that the traffic demand is sufficient to cause queueing on the approach. This condition is termed saturation and the duration is termed saturation interval "ts".

If $\bar{h}cc$ is considered as the mean headway of passenger cars only entering the circulating section, then:

$$\text{Rate of discharge} = \frac{1}{\bar{h}cc} \qquad \ldots\ldots\ldots\ldots\ldots \quad 3.8.1\text{-}1$$

So the total number of passenger cars "N" discharged during the staurated interval "ts" is:

$$N = {}^{ts}/_{\bar{h}cc} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.8.1\text{-}2$$

and the total number of vehicles of different types could be expressed in passenger car units by multiplying each type of vehicle by its PCU factor so $N = b_1X_1+b_2X_2+b_3X_3 \text{ .... } +b_iX_i$ 3.8.1-3

where

b_i = passenger car unit for vehicle type i

X_i = number of vehicles of type i

For the purpose of this study, four categories of vehicles were considered, passenger cars, light goods vehicles, heavy goods vehicles and articulated goods vehicles/ buses.

In the above equation "$\bar{h}cc$" was obtained by analysing a number of groups of cars only entering the circulating section within the traffic flow at the approach lane. The number of each vehicle type was observed during each saturated interval "ts". Then using equation (3.5.3) multiple linear regression analysis was used to calculate PCU factors.

3.8.2 - Computer programming and analysis

The large amount of data obtained from film analysis of roundabout intersections investigated was analysed using computer programming. The statistical Package of the Social Sciences (SPSS) program, described in section (2.7.3) of the previous chapter, was used. The three types of multiple linear regression with variable selection of statistical results were used in the analysis; they are:-

i - Forward method (stepwide) inclusion.
ii - Stepwide Multiple Regression.
iii - Backward Elimination.

Equation (3.8.1-3) was used in which the independent variables were the initial input to the program, in this case the numbers of each vehicle type considered at the saturation interval. The dependent variable is the saturated interval divided by the mean headway of passenger cars. As an example;

$\frac{\text{Saturated interval}}{\text{mean time headway}}$	Cars	LGV	HGV	AGV
10.67	4	2	1	1

Appendix "B" contains data-file samples.

Traffic data were stored in a micro computer during film analysis in the laboratory and then fed into the CYBER computer at the University of Bradford by direct link through interpreting the word "card" to indicate a line of terminal-input.

The output of the first step of the multiple regression is as follows:

(i) $\bar{X}_1$, $\bar{X}_2$, $\bar{X}_3$ ...... $\bar{Y}$, the means of all values for each type of vehicle.

(ii) S_{11}, S_{22}, S_{33} ... the "sample sum of squares", the summation over all data points of the variance squared, used to obtain the sample variance and standard deviation.

(iii) r, r_{in}, ... r_{nn} correlation matrix where r_{ij} is the correlation coefficient between x_i and x_j (the independent variables),

see Appendix C for fully illustrated example.

This output was then utilised in the second step of the multiple linear regression program to determine the actual values of the unknown PCU factors. In this instance, values were required for all four independent variables, but it is possible to obtain values for two, three or more of the vaiables in combination.

As discussed in section (2.7.3) of the previous chapter, the output of the program is mainly the passenger car unit values with their standard error, standard deviation, correlation coefficients and other statistical values to test the significance and the level of confidence in the computed results. Multiple linear regression analysis was also used to test hypotheses about the relationship between a dependent variable and two or more independent variables, and for prediction.

The (t statistics) and "F" value are given in the discussion after each equation. The standard error of the coefficient parameters are given in parenthesis below the parameter in tabular form. R, the coefficient of determination, is used to measure the explanatory power of the regression equation. The explanatory power of the co-efficient is determined by the degrees of confidence by which the test will be accepted; * denotes the 10.00 per cent level of confidence; ** the 5.00 per cent level of confidence; and *** the 1.00 per cent level of confidence. The t statistic for each parameter estimate gives the value of the parameter estimate divided by its estimated standard deviation (the standard error). This value can be compared directly to critical values in the t-table. R^2 is defined as the proportion of the total variation in Y (dependent variable), explained by the regression of Y on X (independent variable). The overall significance of the regression can be tested with the ratio of the explained to the unexplained variance. If the calculated F ratio exceeds the tabular of F at the specified level of significance and degrees of freedom, the hypothesis is accepted.

3.8.3 - Passenger car units in urban situations

The analytical method outlined in section (3.8.1) of this chapter was used to obtain the passenger car unit values for urban situations. The results obtained using the three regression methods discussed in the previous section (3.8.1) of this chapter are tabulated in table (3.8.3-1) for site "1" (Old Street) and table (3.8.3-2) for site "2" (City Road). The computer outputs are given in Appendix "B" with all the statistical values related to the calculation of passenger car unit values.

Using the three regression methods of the package (SPSS) at the University of Bradford CYBER, gave the same values for the PCU factors.

Table (3.8.3-1) and table (3.8.3-2) show the results of the passenger car unit values obtained from Old Street (A5201) and City Road (A501). At the two approaches the observed composition of traffic was as follows;

Vehicle Type	Old Street	City Road
Passenger cars	61%	62%
Light goods vehicle	18%	15%
Heavy goods vehicle	10%	11%
Articulated goods vehicles and double decker buses	11%	12%

The final equation for the first site is:

$$ts/\bar{h}_{cc} = X_1 + 1.15\ X_2 + 2.16\ X_3 + 2.99\ X_4$$

F***(344) ***(75) ***(586) ***(340)

t (18.5) (8.7) (24.2 (18.5)

and for the second site is:

$$ts/h_{cc} = X_1 + 1.14\ X_2 + 2.15\ X_3 + 2.97\ X_4$$

F***(450) ***(79) ***(630) ***(419)

t (21.2) (8.9) (25.1) (20.5)

where X_1, X_2, X_3 and X_4 are the number of the above types of vehicle respectively. The values given in brackets below each factor of the final equation are the F and t values. F and t values were found to be highly significant at the level of 0.1 per cent. This exceeded the tabular value of F which equals 9.12 and t which equals 1.5333.

To test the overall significance of the regression (F) test (overall value) was computed and the result was:

site (1) F = 3629 Significant F = 0.000

site (2) F = 4370 Significant F = 0.000

F values are highly significant at the level of 0.01 per cent. This exceeded the tabular value of F which equals 9.12 at 1 per cent level. Therefore the hypothesis is accepted that the regression parameters are significantly related.

The computed passenger car unit (PCU) values obtained for urban situation update and extend the information related to the characteristics of the various vehicle types. They indicate the difference in the operational capability of each type of vehicle at the entry to the circulating section. Because of the variation in these values traffic composition should be considered as a main factor in the study of intersection capacity and delay to vehicles. Those two factors will be discussed in later sections of this chapter.

SITE 1

Vehicle Type	Passenger Car Unit Values (standard errors are given in brackets)
Passenger Car C	1.00 (0.055)
Light Goods Vehicle LGV	1.15 (0.138)
Heavy Goods Vehicle HGV	2.16 (0.093)
Articulated Goods Vehicle & Buses AGV & B	2.99 (0.168)

Table 3.8.3-1 Computed results of PCU values (Old Street - London)

SITE 2

Vehicle Type	Passenger Car Unit Calues (standard errors are given in brackets)
Passenger Car C	1.00 (0.049)
Light Goods Vehicle LGV	1.14 (0.134)
Heavy Goods Vehicle HGV	2.15 (0.089)
Articulated Goods Vehicle & Buses AGV & B	2.97 (0.151)

Table 3.8.3-2 Computed results of PCU values (City Road - London).

3.8.4 - Passenger car units in semi-rural situations

Passenger car unit values were obtained utilizing the analytical method outlined in section (3.5) of this chapter and are presented in tabular form. The computer outputs are given in Appendix B, showing all the statistical values related to the calculation of passenger car unit factors.

The three regression methods, (see section 3.8.1), used to compute PCU values produced identical results. Table (3.8.4-1) shows the results of the passenger car unit values obtained from the M606/A6036 Rooley Lane roundabout - Bradford.

At this site the observed composition of traffic was as follows;

Passenger cars	63%
Light goods vehicles	14%
Heavy goods vehicles	13%
Articulated goods vehicles	10%

The relationship obtained by regression was;

$$ts/\bar{h}_{cc} = X_1 + 1.16\ X_2 + 1.18\ X_5 + 2.16\ X_4$$

F	***(63.014)	***(14.383)	***(21.97)	***(39.56)
t	(7.94)	(3.8)	(4.7)	(6.3)

where X_1, X_2, X_3, and X_4 are the number of the above types of vehicle respectively. The values given in brackets below each factor of the final equation are the F and t values.

To test the overall significance of the regression, the statistical results were compared with standard statistical values in statistics tables. The overall F value computed was 532.336 which is highly significant at the level of 0.1 per cent of confidence. This exceeded the tabular value of F which equals 9.12 at one per cent level. Over all regression value is 0.9939 and the F and t computed and tabular values are given below at 1 per cent level of significance for each vehicle type PCU factor;

Vehicle Type	F(computed)	F(tabular)	T(comp.)	T(tabular)
passenger car	63	9.12	7.938	1.533
light goods vehicles	14.38	9.12	3.79	1.533
heavy goods vehicles	21.98	9.12	4.69	1.533
articulated goods vehicles	39.56	9.12	6.29	1.533

The results showed that the relationship between the ratio $ts/\bar{h}_{cc}$ in equation (3.8.1-2) and the total number of vehicles converted to PCU's is significant and positively related.

On the basis of the results, it is reasonable to conclude that the adopted method for estimating passenger car unit values will provide a satisfactory means for the study of vehicle performance in the operating conditions of roundabouts.

Table (3.8.4-2) gives a summary of the computed values of passenger car units. It can be seen that passenger car unit values for heavy and articulated goods vehicles at sites (1) and (2) are higher than those at site (3) which indicates their greater effects in the urban situation than in the semi-rural situation. This is because of the difference in the geometric features of the sites which provide different manoeuvring facilities.

Finally the drivers in the urban situation must observe speeds which are normally lower than those observed by drivers in the semi-rural situation.

Passenger car unit values at sites (1) and (2) are 1.15, 2.16 and 1.98 for light, heavy and articulated goods vehicles respectively. Passenger car unit values at site (3) are 1.16, 1.86 and 2.16 for light, heavy and articulated goods vehicles.

SITE 3

Vehicle Type	Passenger Car Unit Values (standard errors are given in brackets)
Passenger Car C	1.00 (0.139)
Light Goods Vehicle LGV	1.16 (0.339)
Heavy Goods Vehicle HGV	1.86 (0.442)
Articulated Goods Vehicle & Buses AGV & B	2.16 (0.382)

Table 3.8.4-1 Computed results of PCU values (M606)

Vehicle Type	Mean PCU Values (Standard errors are in brackets)		
	Site (1)	Site (2)	Site (3)
Passenger Car C	1.00 (0.055)	1.00 (0.049)	1.00 (0.139)
Light Goods Vehicle LGV	1.15 (0.138)	1.14 (0.138)	1.16 (0.339)
Heavy Goods Vehicle HGV	2.16 (0.093)	2.15 (0.089)	1.86 (0.442)
Articulated Goods Vehicle & Buses AGV & B	2.99 (0.168)	2.97 (0.151)	2.16 (0.382)

Table 3.8.4-2 Summary of computed results of PCU values for the 3 sites.

3.9 - The effect of vehicle type on entry capacity

3.9.1 - Introduction

The capacity of an intersection is of prime importance in the study of its performance. Using the data obtained for the geometric and traffic parameters the relationship between entry/circulating flow have been investigated. The investigation was carried out by calculating entry flow "Q_e" (pcu/h) and circulating flow Q_c (pcu/h). These values were obtained by using the geometric parameters for each intersection and by using the computed passenger car units. The relationships between the two parameters were then plotted. Since the capacity of the roundabout is affected by traffic composition, vehicle type effects on entry/circulating flow performance were determined using vehicle type percentages and their PCU values, where entry/circulating flows are expressed in pcu/h. For each approach percentages of vehicle types were recorded during the investigation at peak hours. Also maximum observed flows were obtained. Using the unified formula (92) to calculate roundabout capacity, predicted and observed flows were compared to show the effect of vehicle types.

3.9.2 - Vehicle type/entry - circulating flow relationship

At the three sites, observations of entry and circulating flows were carried out for a total period of six hours each during morning and afternoon peak hours. Heavy and articulated goods vehicles were combined and were expressed as a percentage of the total flow both for entry and circulating flows. The variations in both entry and circulating flows corresponding to the percentages of the above types of vehicles were plotted and shown in figures (3.9.2-1) to (3.9.2-6) for sites 1, 2 and 3 respectively.

The figures show that both entry and circulating flows are affected by the increase in the percentages of buses, heavy and articulated goods vehicles. Entry and circulating flows are reduced when the percentage of heavy and articulated goods vehicles increases. This indicates higher PCU values for heavy vehicles compared with passenger cars.

Sites (1) and (2) show agreements in the way that the percentage of heavy and articulated goods vehicles affects the entry/circulating flows whereas site (3) shows a different pattern. In this case, the increased percentage of heavy vehicles has a less significant effect on entry/circulating flows.

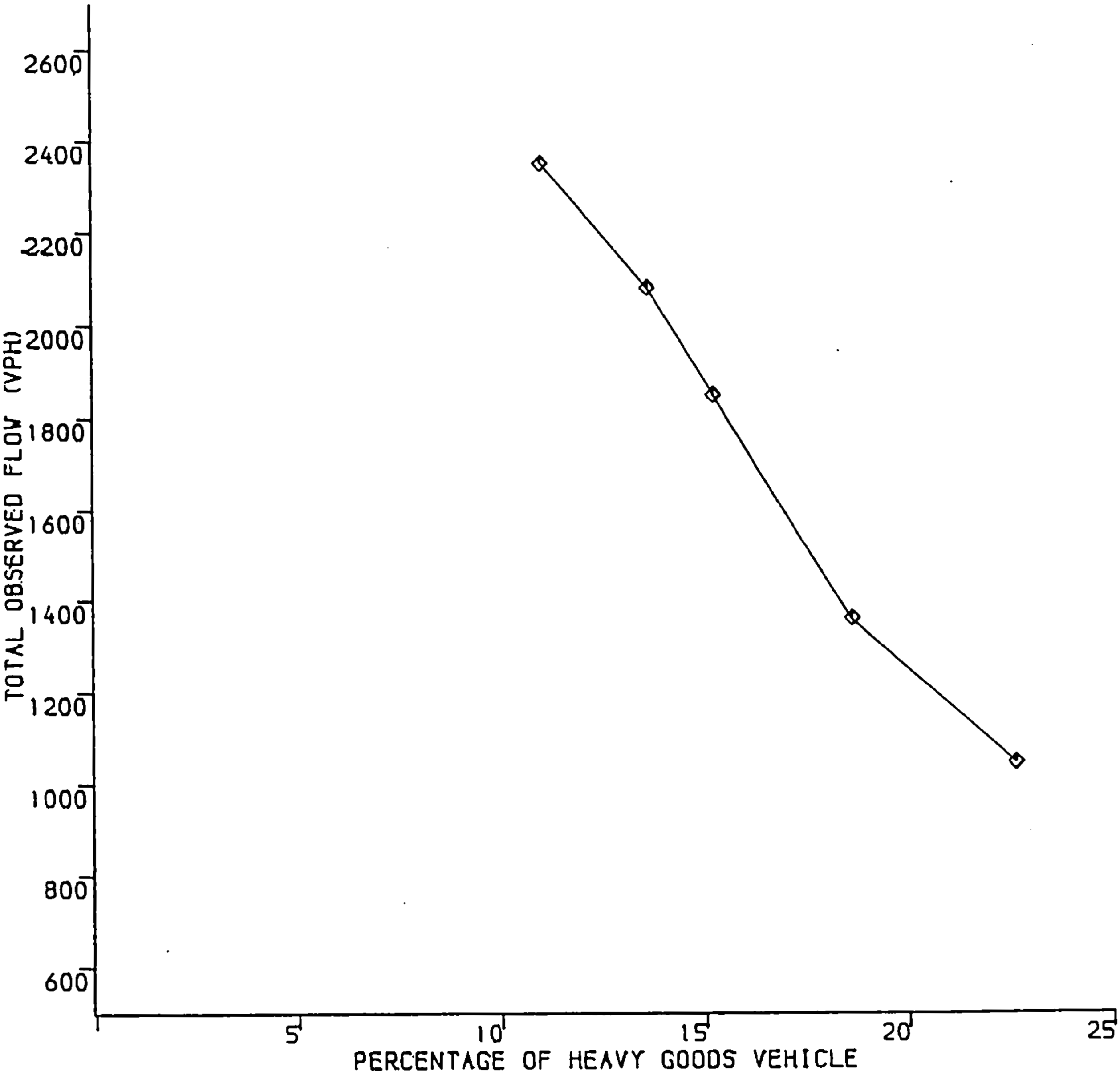


Figure 3.9.2-1 Observed entry flows and percentages of heavy and articulated goods vehicles relationships. Old Street - London.

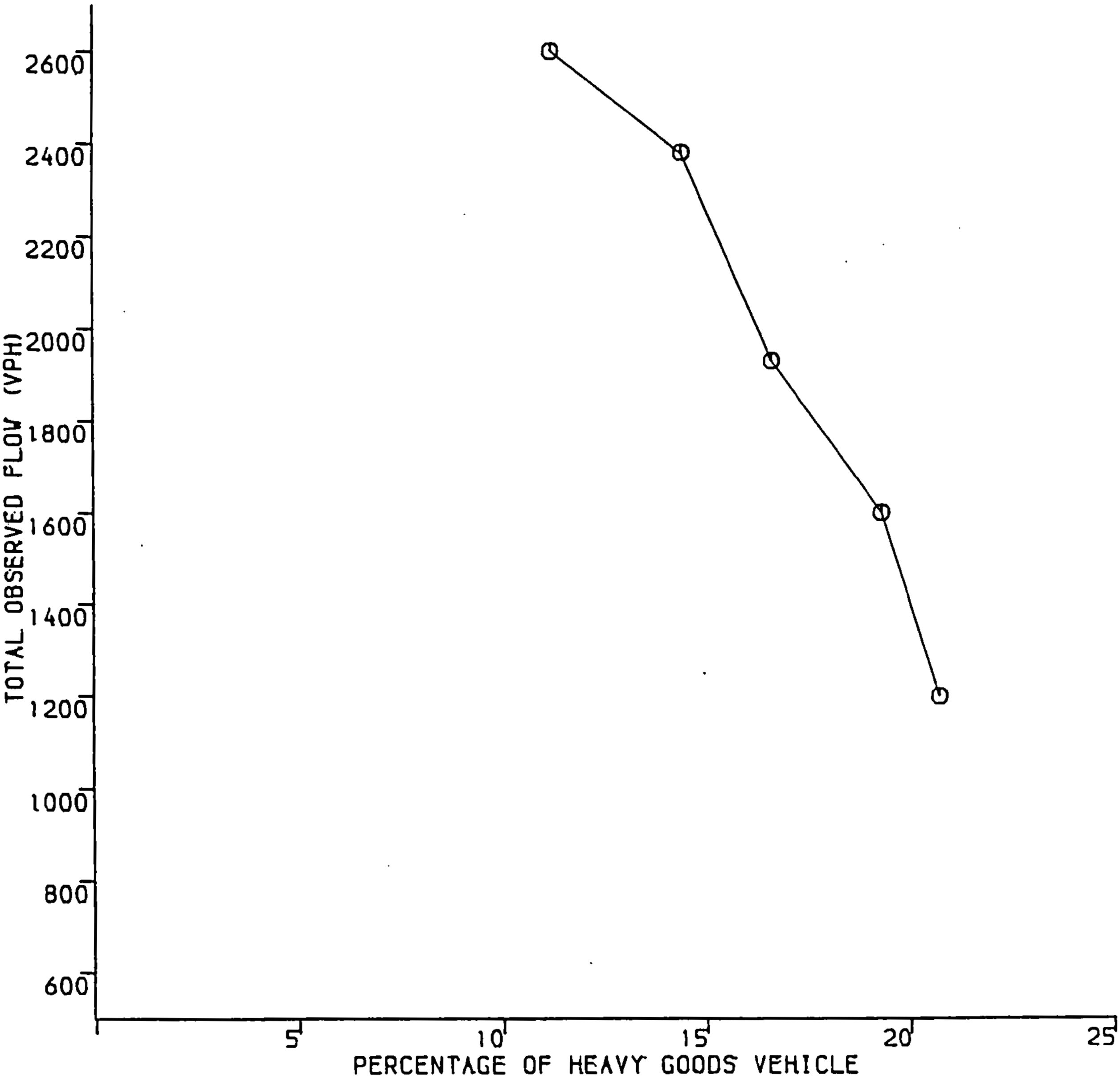


Figure 3.9.2-2 Observed circulating flows and percentages of heavy and articulated goods vehicles relationships. Old Street - London.

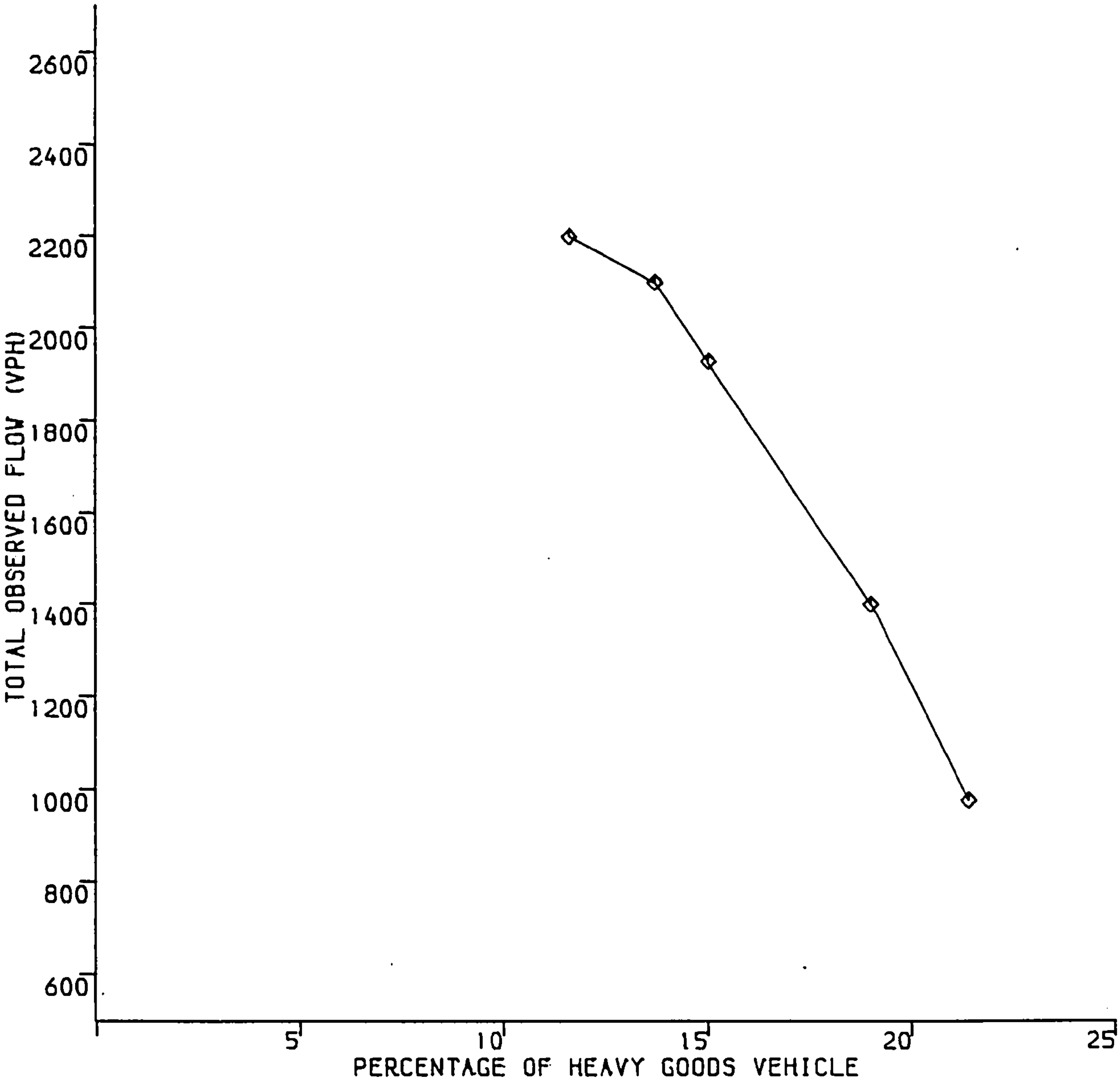


Figure 3.9.2-3 Observed entry flows and percentages of heavy and articulated goods vehicles relationships. City Road - London.

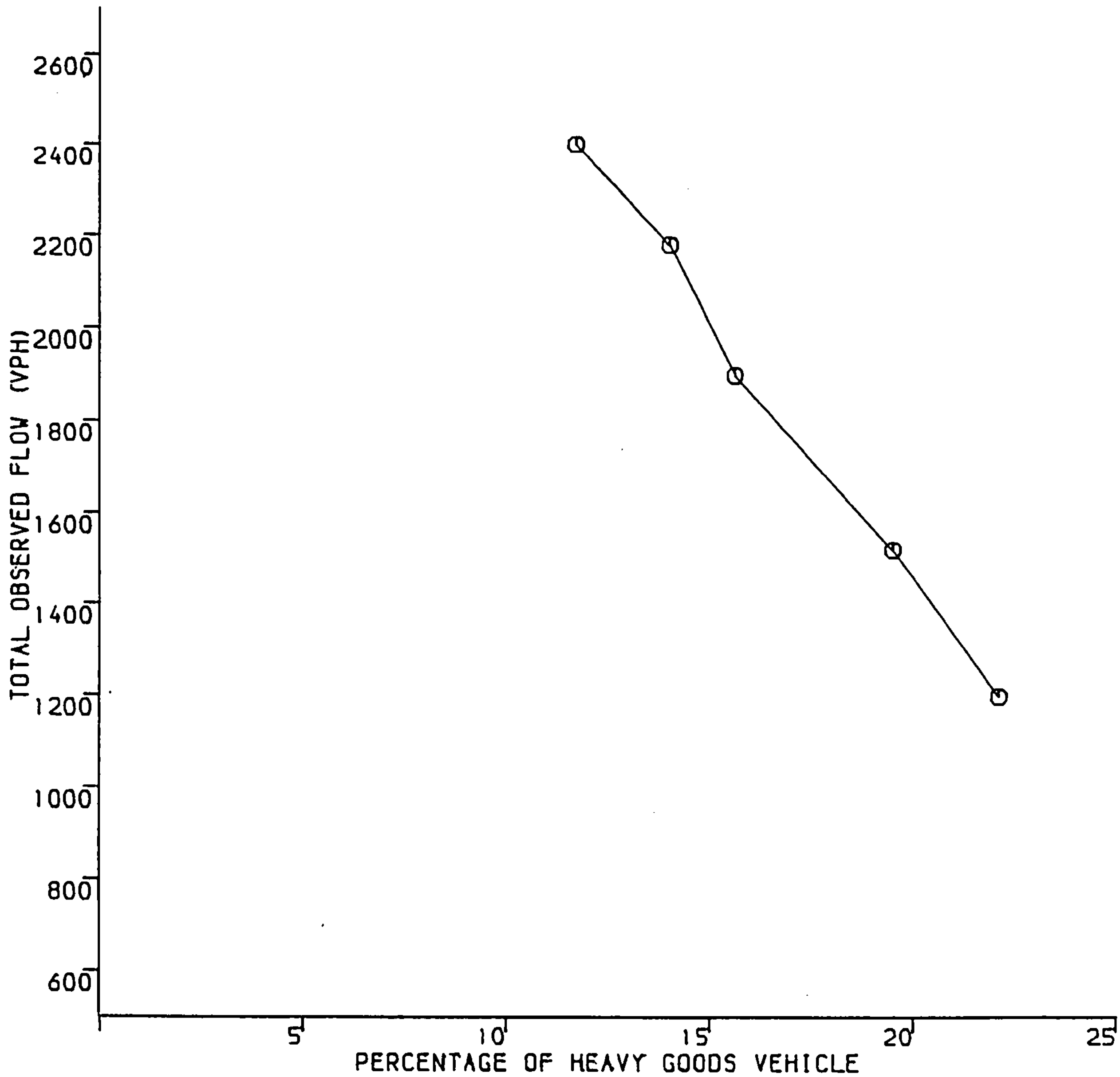


Figure 3.9.2-4 Observed circulating flows and percentages of heavy and articulated goods vehicles relationships. City Road - London.

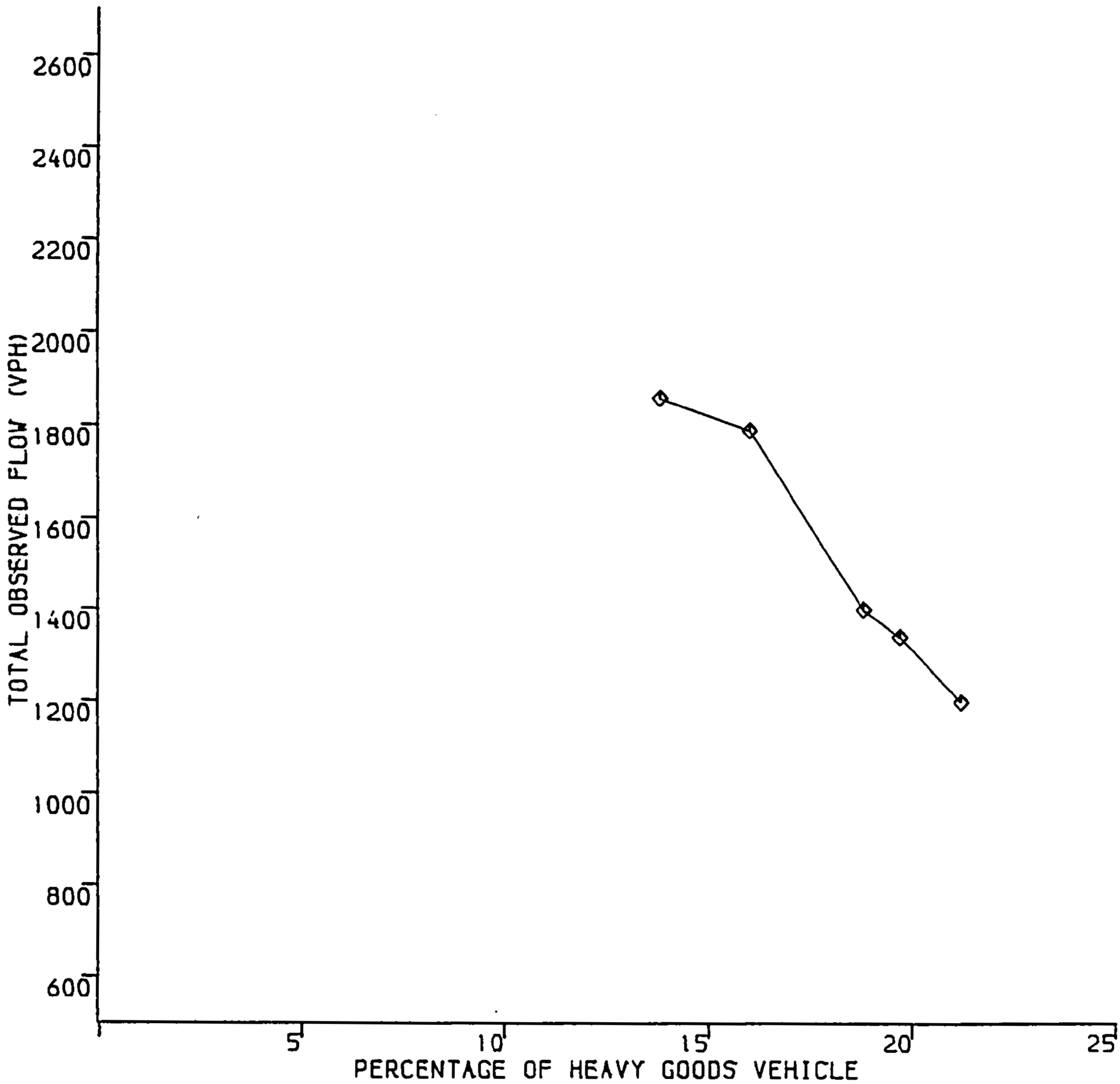


Figure 3.9.2-5 Observed entry flows and percentages of heavy and articulated goods vehicles relationships. M606 Motorway.

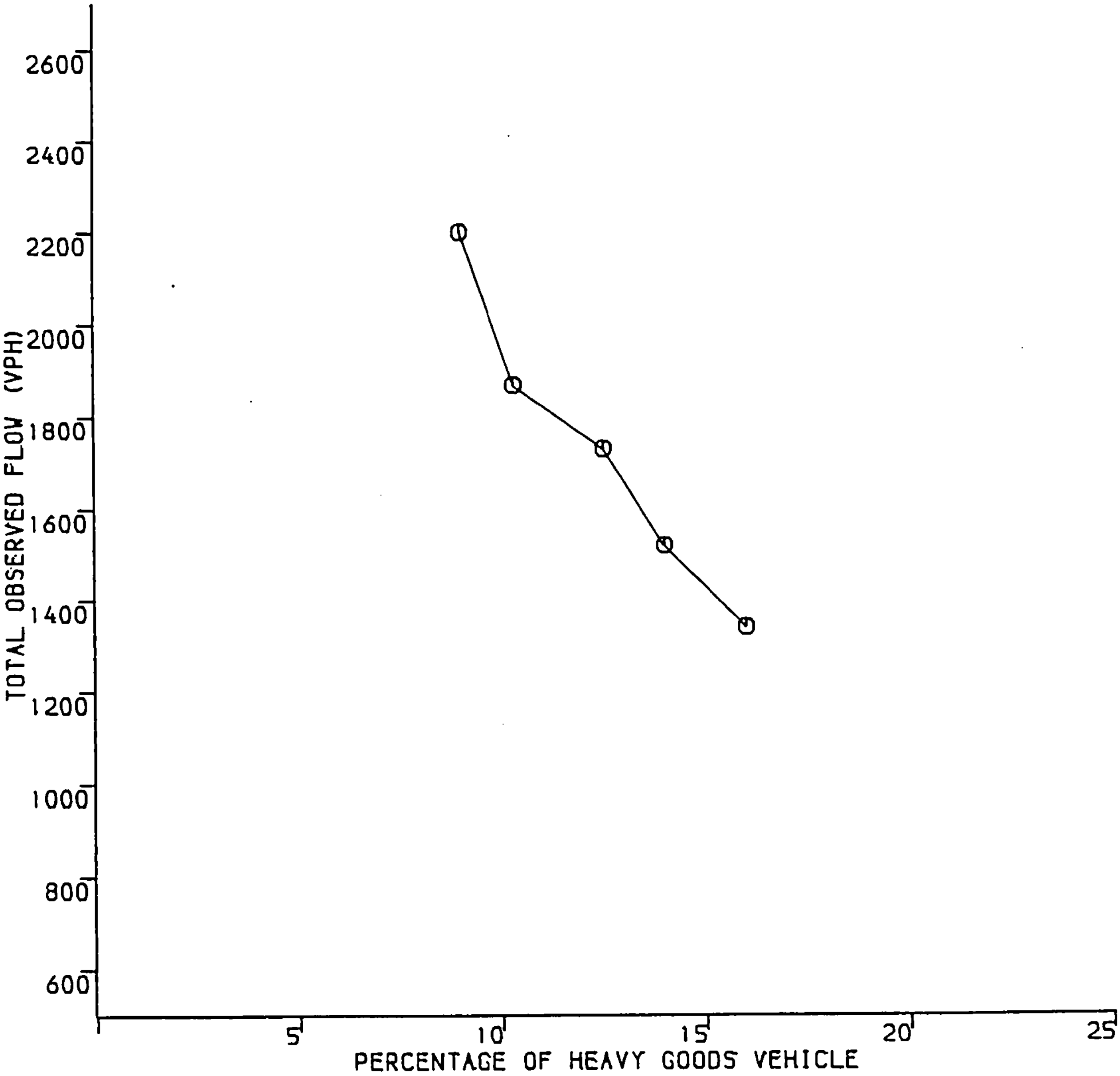


Figure 3.9.2-6 Observed circulating flows and percentages of heavy and articulated goods vehicles relationships. M606 Motorway.

3.9.3 - Comparison with the TRRL unified capacity formula

The sites chosen for the study as mentioned before were required to have continuous queueing for periods of at least forty minutes during the morning or the afternoon peak hours. Their geometric parameters are as shown in table (3.9.3-4). The diagram of the approaches under consideration are as shown in figures (3.5.3-1) to (3.5.3-5).

To investigate the capacity of the approaches' total entry widths were considered to calculate entry flows. Measurements were made of "Q_e", the saturated entry flow (i.e. the flow entering the roundabout when there was queueing in both approach lanes) and of "Q_c", the corresponding circulating flow.

Observations were taken for consecutive saturated intervals of six minutes during peak periods. Vehicles were classified as "passenger cars", "light goods vehicles", "heavy goods vehicles" and "articulated goods vehicles".

The sites' geometric parameters were used to calculate the factors "F" and "f_c" in the unified formulae for urban and semi-rural situation (92). The factors are;

For site (i)

$Q_e = F - f_c Q_c$ pcu/h

$F = 3220.86$

$f_c = 0.682$

$Q_e = 3220.86 - 0.682 \times Q_c$ pcu/h

For site (2)

$Q_e = F - f_c Q_c$ pcu/h

$F = 3087.9$

$f_c = 0.645$

$Q_e = 3087.9 - 0.645 \times Q_c$ pcu/h

For site (3)

$Q_e = 1.11F - 1.40\ f_c Q_c$

$F = 2276.6$

$f_c = 0.531$

$Q_e = 2527.03 - 0.743 \times Q_c$ pcu/h

The observed entry and circulating flows were calculated using the computed passenger car units in the previous section of this chapter. They are:

Type of vehicle	Passenger car unit values Site (1)	Site (2)	Site (3)
Cars	1.00	1.00	1.00
LGV	1.15	1.14	1.16
HGV	2.16	2.15	1.86
AGV	2.99	2.97	2.16

The observed entry and circulating flows shown in tables (3.9.3-1), (3.9.3-2) and (3.9.3-3) for sites (1), (2) and (3) respectively, were selected from four highly saturated intervals of 6 minutes duration for each site. They are converted to passenger car units per hour using the above computed values and the percentage of each vehicle type observed.

Circulating Flows		Entry Flows	
VPH	PCU/H	VPH	PCU/H
1320	1797	2110	2874
1460	1979	2061	2807
1122	1521	2230	3035
1455	1981	2012	2739

Table 3.9.3-1 Observed circulating and entry flows, Old Street - London.

Since many public road junctions carry different percentages of heavy commercial vehicles with a wide range of sizes and combination ranging from articulated goods vehicles and double decker buses to twin and triple trailers which operate under conditions of imbalanced demand, therefore vehicle type effects vary at different junctions depending on junction layouts, number of arms and traffic demand.

The observed flows at the sites investigated were

Circulating Flows		Entry Flows	
VPH	PCU/H	VPH	PCU/H
968	1397	1920	2703
1242	1748	1822	2564
1114	1513	1896	2668
1436	2020	1781	2393

Table 3.9.3-2 Observed circulating and entry flows, City Road - London.

Circulating Flows		Entry Flows	
VPH	PCU/H	VPH	PCH/H
1223	1546	1407	1778
1462	1878	1280	1617
1322	1668	1396	1809
1527	1961	1087	1385

Table 3.9.3-3 Observed circulating and entry flows, M606 Motorway - Bradford.

plotted and compared with values obtained by the application of the unified formulae developed by the Transport and Road Research Laboratory. Figures (3.9.3-1), (3.9.3-2) and (3.9.3-3) show observed and predicted entry/circulating flow relationships for sites (1), (2) and (3) respectively.

In order to show the effect of traffic composition at each site, the following results were obtained from figures (3.9.3-1) to (3.9.3-3).

For a value of 1,700 pcu/h of circulating flow the differences between observed and predicted entry flows were:

At site (1)	880	pcu/h
At site (2)	590	pcu/h
At site (3)	485	pcu/h

The differences arise from the variation in the percentages of heavy commercial vehicles at each site, which is related to the findings in section (3.9.2) of this chapter. Also it is due to the variations in the entry widths of the approaches and to the difference in use of the circulating carriageway by different vehicle types.

Site Number	Site location	V(m)	e(m)	D(m)	r(m)	ℓ'(m)	Φ°
(1)	Old Street London	11	11.60	80	9	2	30
(2)	City Road London	11	8.0	80	12	2	60
(3)	M606 Bradford	7.30	7.75	95	32	2.5	38

Table 3.9.3-4 The sites studied, with values of their geometric parameters.

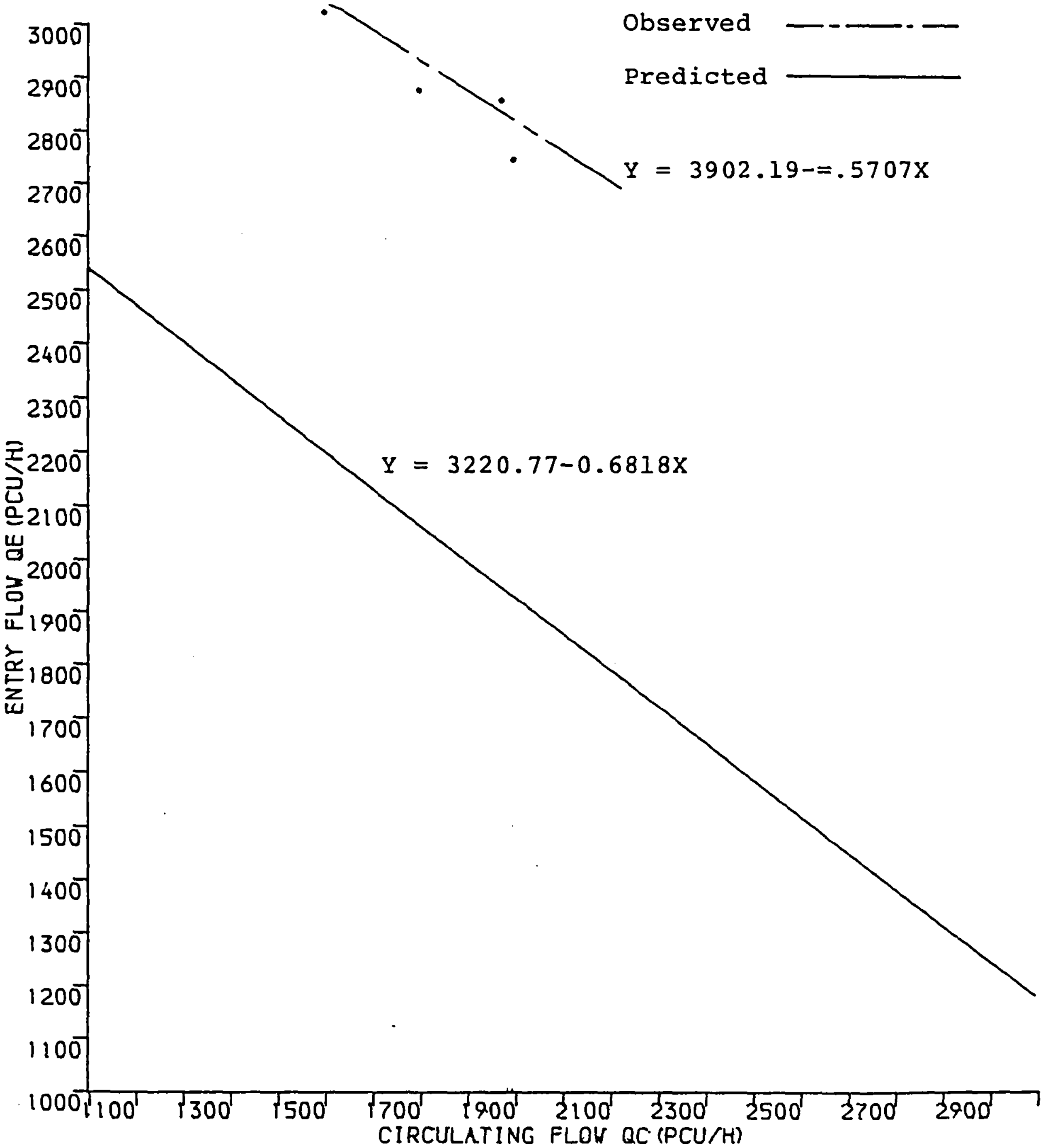


Figure 3.9.3-1 Observed and predicted flow relationships. Site 1.

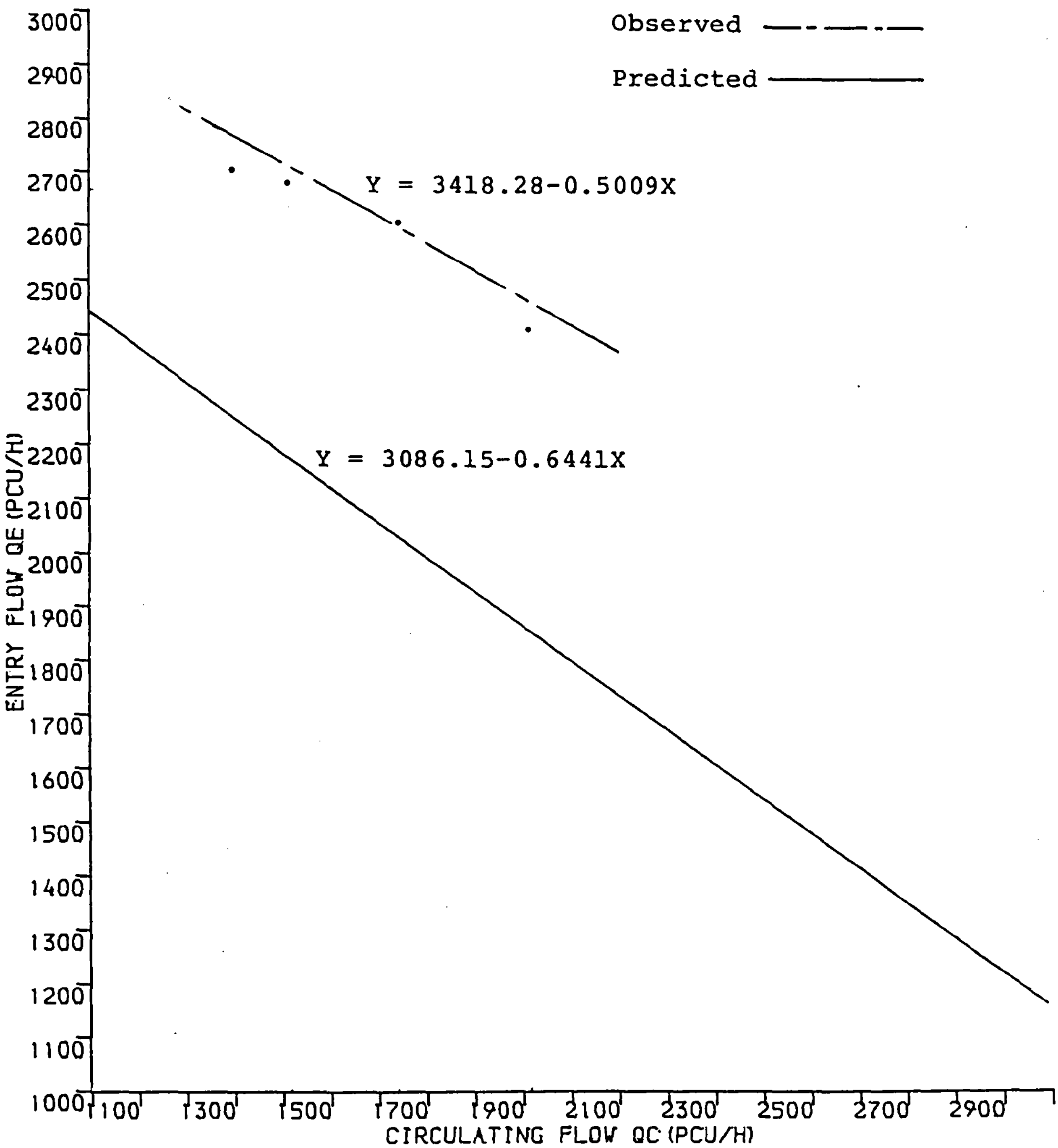


Figure 3.9.3-2 Observed and predicted relationships.
Site 2.

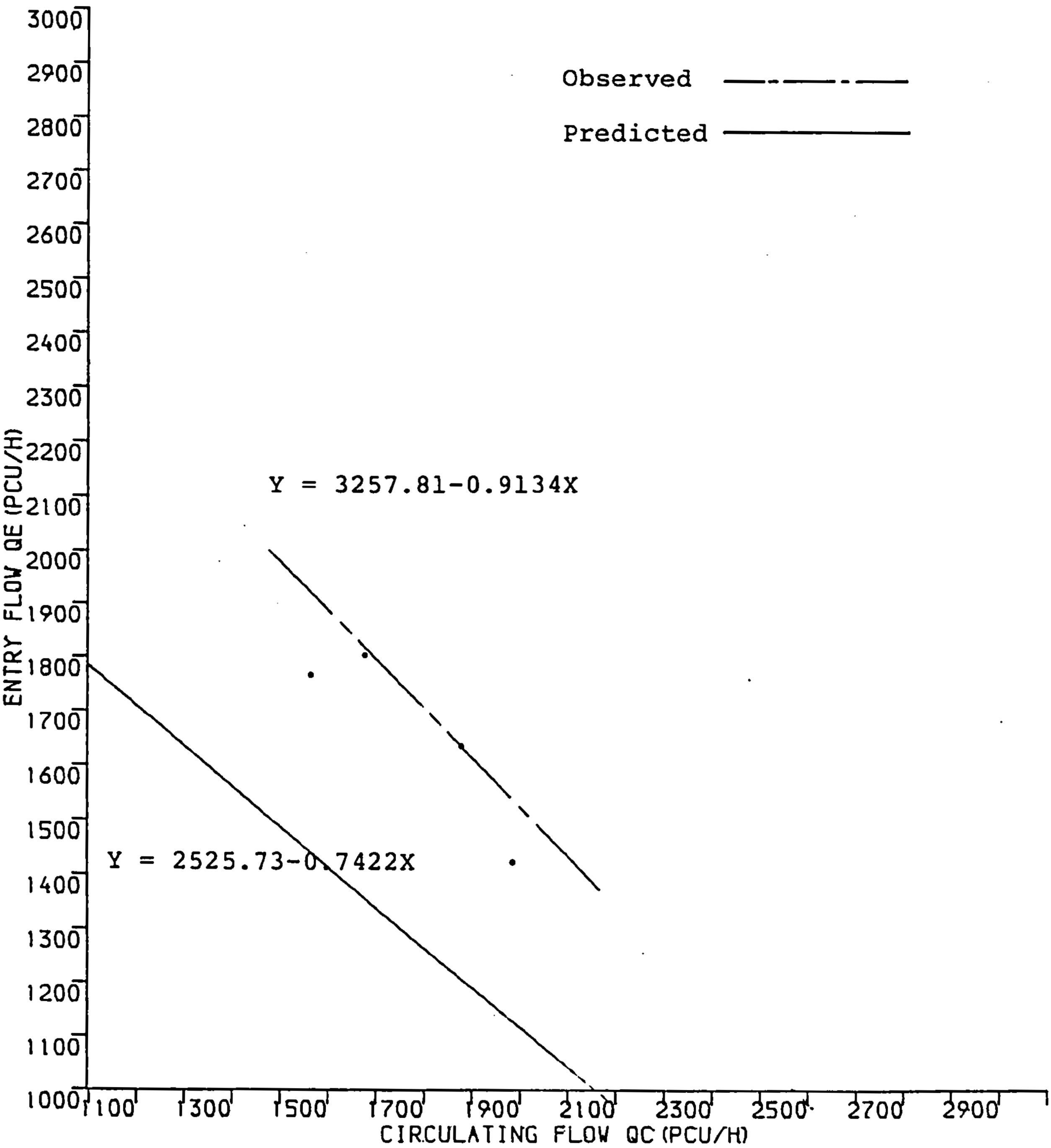


Figure 3.9.3-3 Observed and predicted relationships. Site 3.

3.10 - Effect of vehicle type on delay

Vehicle delay is generally viewed by traffic engineers as a measure of intersection performance. The variation in vehicle type characteristics and percentage is considered in this study as one of the traffic factors that affect delay. In this study the approach traffic flow is analysed by grouping vehicles into four types (passenger cars, light goods vehicles, heavy goods vehicles and articulated goods vehicles). The values of mean gap acceptance for each vehicle type are:

Location	Vehicle type	Gap mean acceptance (seconds)	% of vehicle type	Combined mean gap acceptance (seconds)
Site (1)	Cars	22.42	61%	
Old Street	LGV	3.715	18%	3.20
	HGV	5.269	10%	
	AGV	5.802	11%	
Site (2)	Cars	3.108	62%	
City Road	LGV	3.977	15%	3.80
	HGV	5.426	11%	
	AGV	5.669	12%	
Site (39	Cars	3.296	63%	
M606	LGV	3.637	14%	3.52
	HGV	3.969	13%	
	AGV	4.141	10%	

According to the latest design criteria of the Transport and Road Research Laboratory, roundabouts could be treated as a series of T-junctions. The junctions are of priority type in which all vehicles are turning left.

Therefore the average delay to entering vehicles could be calculated using the proposed formula by Tanner (102).

The variables used to calculate average delay in this formula are:

1 - The circulating volume q_1

2 - The entry volume q_2

3 - The minimum time headway between circulating vehicles β_1

4 - The minimum time headway between entry vehicle β_2

5 - The average lag or gap "α" in the circulating flow accepted by entry flow drivers.

The circulating flow for the investigation is a fixed value of 700 vehicles per hour and theentry flow is to vary from 100 to 500 vehicles per hour in 50 vehicles per hour step increase. The procedure to find mimimum time headway in both the entry and circulating flows is to consider a bunch or platoon of vehicles following each other with minimum separation. The mean lag or gap "α" for each individual vehicle as obtained in section (3.8) of this chapter is used. Also the combined mean gap acceptance is used for comparison purposes using the percentage of each vehicle type at each site for the calculation of delay.

Tanner (102) proposed that the average delay to minor road vehicles $\bar{w}_2$ could be determined from,

$$\bar{w}_2 = \frac{\frac{1}{2}E(Y^2)/Y + q_2\, Y \exp(-\beta_2 q_1)\left[\exp(\beta_2 q_1) - \beta_2 q_1 - 1\right]/q_1}{1 - q_2\, Y\left[1 - \exp\,(-\beta_2 q_1)\right]}$$

$$E(Y) = \frac{\exp\left[q_1\ (\alpha - \beta_1)\right]}{q_1\ (1 - \beta_1 q_1)} - \frac{1}{q_1}$$

$$E(Y^2) = \frac{2\ \exp\left[q_1(\alpha - \beta_1)\right]}{q_1^{\,2}(1 - {}_1 q_1)^2}\left[\exp\left[q_2\ (\alpha - \beta_1)\right] - \alpha q_1(1 - \beta_1 q_1)\right.$$

$$\left. - 1 + \beta_1 q_1 - \beta_1^{\,2} q_1^{\,2} + \tfrac{1}{2}\beta_1^{\,2} q_1^{\,2}/(1 - \beta_1 q_1)\right]$$

$$Y = E(Y) + 1/q_1$$

The results of average delay for each site are tabulated and given in tables (3.10.1) to (3.10.2).

The first stage of analysis is to consider 100% of a particular vehicle type present in the approach traffic flow. The mean gap acceptance for each type is then used to calculate average delay. The second stage is to use the combined mean gap acceptance in the calculation of average delay.

Figures (3.10.1) to (3.10.3) show the variation in average delay to vehicles according to vehicle type and traffic volume.

The results indicated that average delay to vehicles increases with the percentages of heavier vehicles present in the traffic flow. This is because of their ability to manoeuvre.

The results of average delay to vehicles obtained by using the combined mean gap acceptance are found to have values closer to those of the highest percentage of vehicle type present in the traffic stream. The results indicated the significance of gap acceptance mean values on average delay which reflect the effect of different vehicle types on delay as they vary in their gap acceptance.

As it has been shown by graphs, the delay values obtained using Tanners equation show an agreement between sites (1) and (2). Heavy and articulated goods vehicle show higher values of delay compared with those of cars, light goods vehicles and combined flow. At site (3) delay values show less effect and lower values than average delay at sites (1) and (2), which indicates the significant effects of vehicle types at urban sites.

Entry flow v/h	Circulating flow v/h	Average delay (seconds)				
		100% Cars	100% LGV	100% HGV	100% AGV	Combined
100	700	1.6	3.4	7.0	8.8	2.60
150	"	1.8	3.9	8.3	10.7	3.00
200	"	2.0	4.5	10.1	13.3	3.42
250	"	2.3	5.3	12.6	17.4	3.95
300	"	2.6	6.2	16.6	24.6	4.60
350	"	3.0	7.5	23.6	40.6	5.43
400	"	3.5	9.3	39.6	107.8	6.53
450	"	4.1	12.1	111.9	-	8.04
500	"	4.9	16.7	-	-	10.25

Table 3.10.1 Average delay to entry traffic flow (Old Street - London)

Entry flow v/h	Circulating flow v/h	Average delay (seconds)				
		100% Cars	100% LGV	100% HGV	100% AGV	Combined
100	700	2.5	3.88	7.52	8.35	3.56
150	"	2.8	4.46	8.95	10.00	4.08
200	"	3.3	5.17	10.9	12.40	4.71
250	"	3.8	6.08	13.80	16.00	5.51
300	"	4.40	7.28	18.50	22.10	6.55
350	"	5.10	8.93	27.30	34.90	7.95
400	"	6.10	11.35	49.70	77.60	9.94
450	"	7.50	15.24	232.60	-	13.00
500	"	9.50	22.51	-	-	18.31

Table 3.10.2 Average delay to entry flow (City Road - London)

Entry flow v/h	Circulating flow v/h	Average delay (seconds)				
		100% Cars	100% LGV	100% HGV	100% AGV	Combined
100	700	2.76	3.28	3.86	4.20	3.10
150	"	3.14	3.75	4.44	4.83	3.53
200	"	3.60	4.30	5.10	5.60	4.06
250	"	4.20	5.00	6.10	6.70	4.72
300	"	4.90	5.90	7.20	8.00	5.55
350	"	5.80	7.20	8.90	10.00	6.64
400	"	7.00	8.80	11.30	12.90	8.13
450	"	8.60	11.30	15.10	17.80	10.30
500	"	11.10	15.40	22.30	27.90	13.71

Table 3.10.3 Average delay to entry traffic flow (M606 - Bradford)

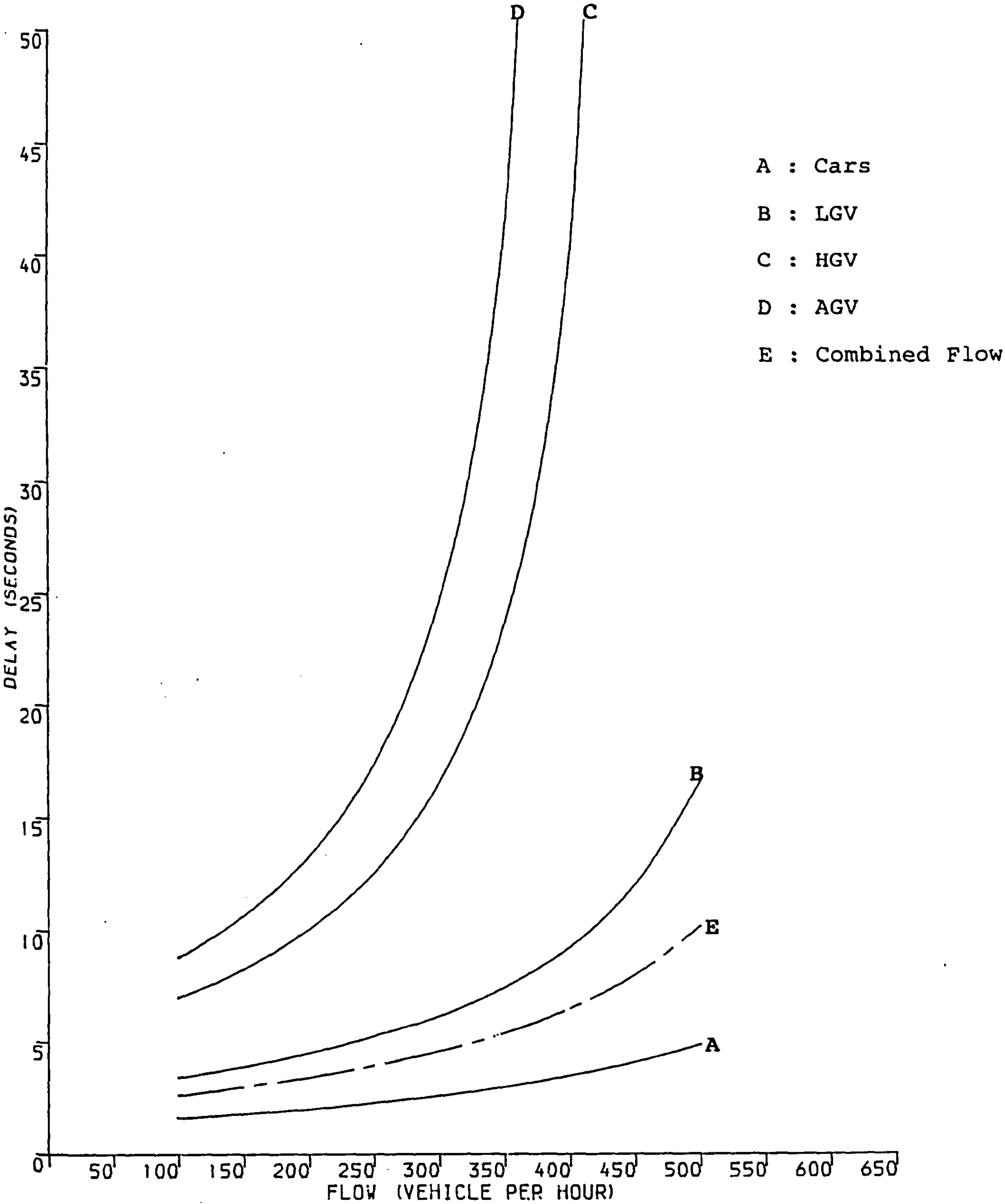


Figure 3.10.1 Average delay to entry traffic flow (Old Street - London).

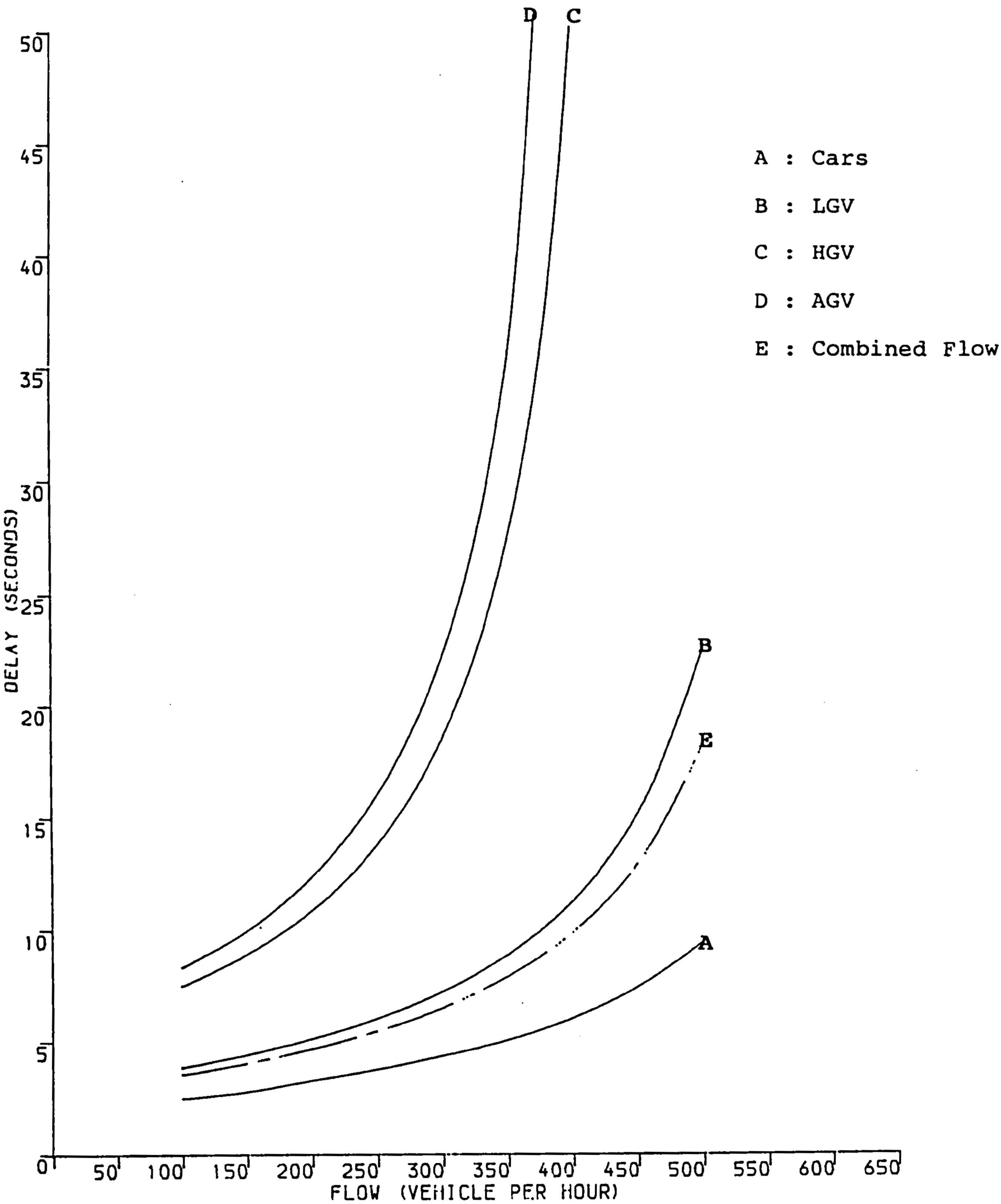


Figure 3.10.2 Average delay to entry traffic flow (City Road - London).

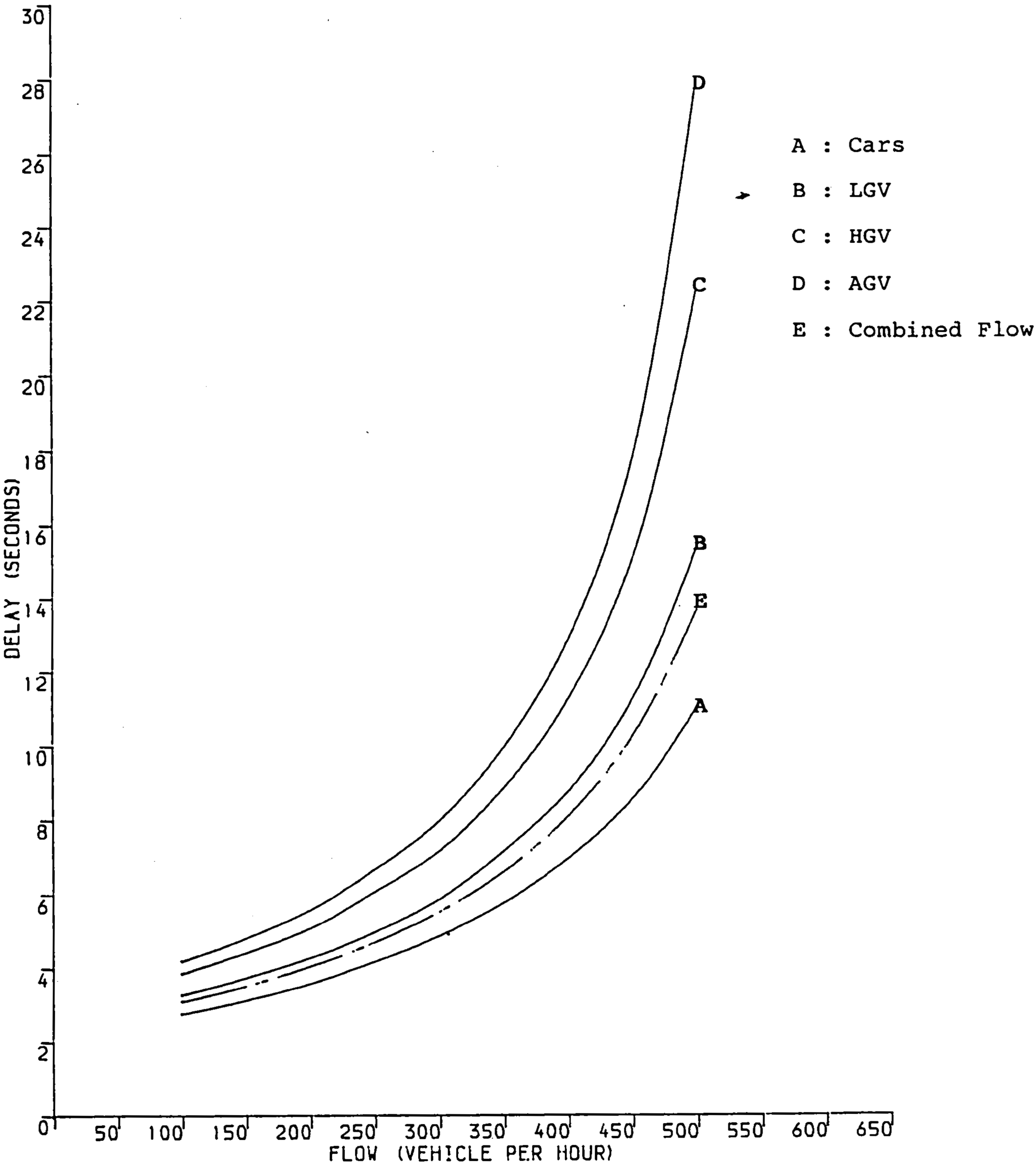


Figure 3.10.3 Average delay to entry traffic flow (M606 - Bradford).

4

Effects of Vehicle Type on Rural Motorway Flow

CHAPTER FOUR

Effect of vehicle type on rural motorway flow

4.1 - Introduction and summary

Vehicle type, design and performance are factors affecting capacity, safety and level of services on rural and urban roads. The increase of heavy commercial vehicles has a great influence on the operation of other vehicles and is a growing concern of highway designers. It is the aim of this chapter to study some of the effects of vehicle type on highway traffic flow, expressed in their passenger car unit equivalencies and speed characteristics.

4.2 - Speed distribution studies

4.2.1 - Introduction

Speed, flow and concentration are the fundamental parameters when describing traffic flow. Speed may be defined as the rate of travel or the rate of movement of a vehicle, usually expressed in kilometres per hour (km/h) or in miles per hour (mph) and is generally qualified according to three main types:

(a) Spot speed: the instantaneous speed of a vehicle at any specified point.

(b) Running speed: the average speed maintained over a particular section of a highway on which the vehicle is moving, (calculated by dividing length of section of highway by time vehicle is in motion, not including stopped delays).

(c) Journey speed: the effective speed of the vehicle on a journey between two points (found by dividing distance between the two points by total time taken to complete journey, including any stopped time due to traffic delays).

In a typical journey, where stopping delays are incurred, then the journey speed must be slower than the running speed and spot speeds will vary from zero to maximum, some being in excess of the running speed. High running speeds with low journey speeds are undesirable and represent stop-go conditions with enforced decelerations and accelerations. Uniformity between the two speed measures denotes comfortable travel conditions. Drivers expect a higher speed on a motorway than on an urban arterial, so when speed is used as a measure of effectiveness, speed criteria must recognise driver expectations, roadway function and vehicle type performance especially on gradients.

4.2.2 - Speed-flow variation by hour of day

As the number of vehicles occupying a roadway increases there is an associated decrease in speed especially at flow rates approaching capacity, under ideal conditions. Speed varies according to the time of day and day of the week and is affected by type of facility and vehicle type percentage in the total flow, especially heavier ones. Figure (4.2.2-1) illustrates variations of speed with time of day, along with hourly volume variations, over a 24-hour period during weekdays in the U.S.A. Figure (4.2.2-2) shows variation patterns during weekend periods. From the two figures it is clear that speed varies with the volume of flow and shows a marked response when the volume exceeds approximately 1,600 vphpl. The speeds in figure (4.2.2-1) and (4.2.2-2) are virtually the same, despite significantly lower volumes at weekends. This is a reflection of driver populations and trip purpose impacts. Weekend drivers may be less familiar with the facility, or if familiar, do not drive with the same sense of urgency devoted to their daily commuting to work.

Recent data on speed trends shows a more gradual increase in the countries where speed limits are in operation. Data from the U.K.(104) and U.S.A.(103) shows an increase of up to 10 mph (16 km/h) and the general speed increase level is higher on rural arterials than on other urban roads. This

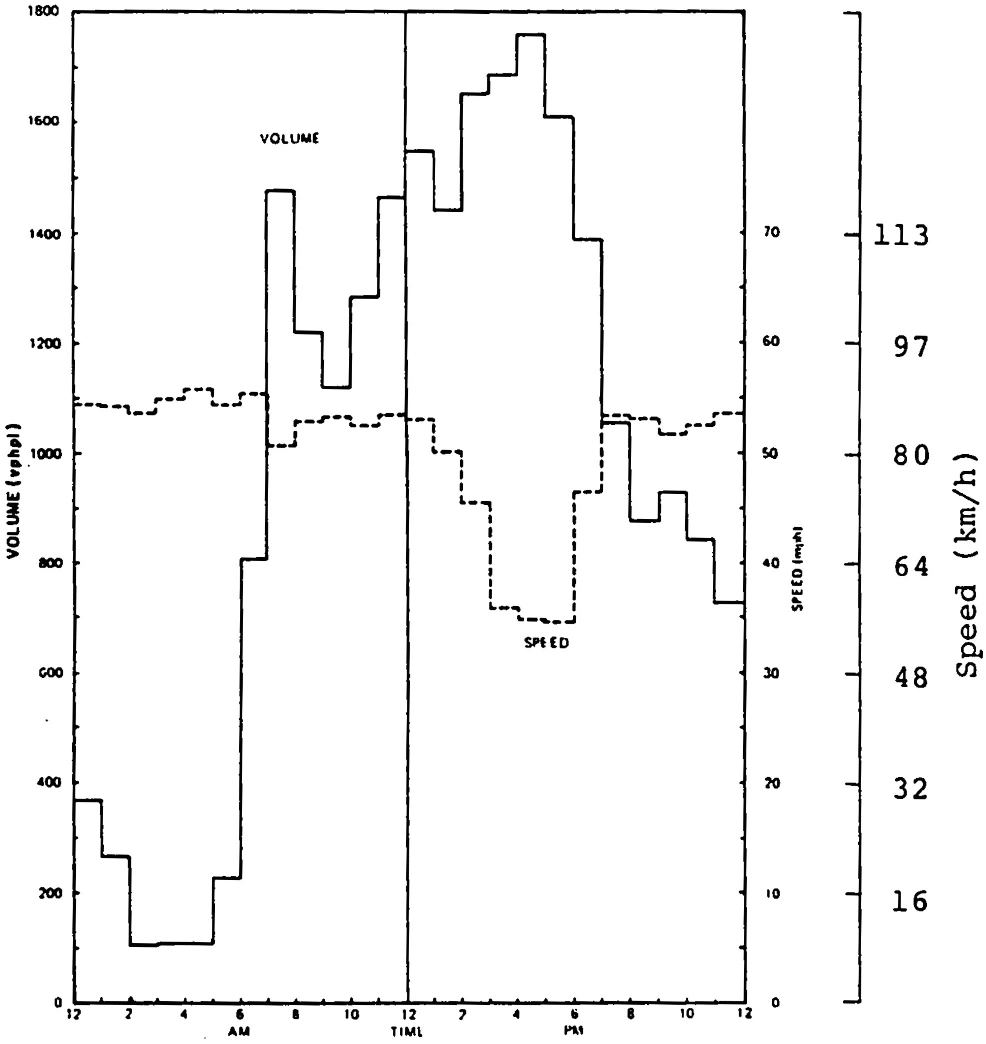


Figure 4.2.2-1 Speed variation by hour of day during week days.
(Reproduced from reference No.103)

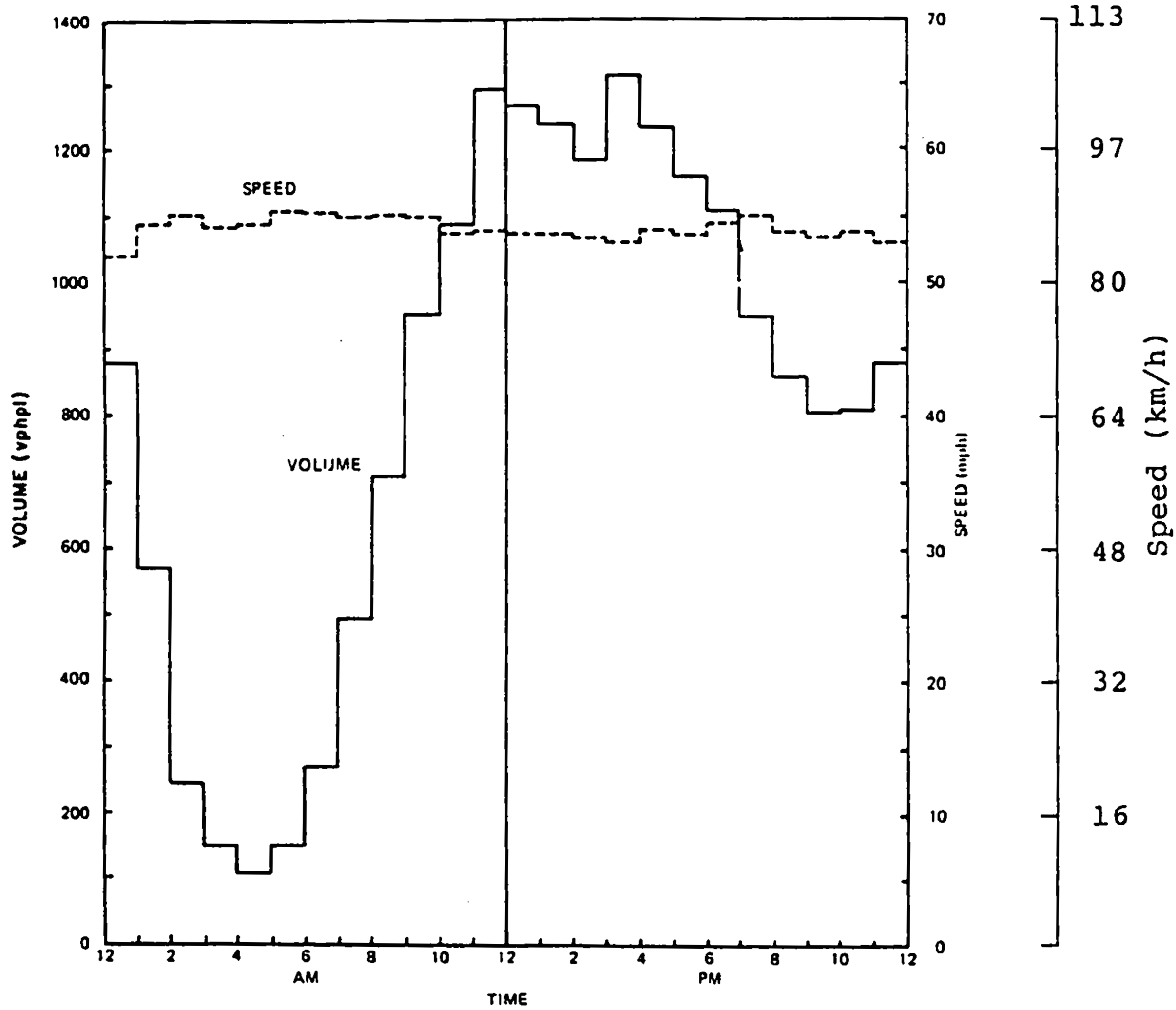


Figure 4.2.2-2 Speed variation by hour of day, Saturdays. (Reproduced from reference No.103)

reflects the better design of both highways and vehicles throughout the last decade.

4.2.3 - Speed-flow relationship for multilane highways

A knowledge of the relationships between speed, flow (volume), and capacity is basic to the understanding of the place of capacity in the design of highways and their operational characteristics. Figures (4.2.3-2) and (4.2.3-3) give the relationships for a single freeway or expressway lane and are used to show these relationships at differing level of services (103).

If a single vehicle travels along a traffic lane, the driver is free to proceed at the design speed. This situation is represented at the beginning of the appropriate curve at the upper left of figure (4.2.3-2). But as the number of vehicles in the lane increases, the driver's freedom to select speed is restricted. This restriction brings a progressive reduction in speed. For example, many observations have shown that for a highway designed for 70 mph (113 km/h), when flow reaches 1900 passenger cars per hour, traffic is slowed down to about 43 mph (69 km/h). If flow increases further, the relatively stable normal flow condition usually found at lower volumes is subject to break-down. This zone of instability is shown by the shaded area on the right-hand side of figure (4.2.3-1). One possible consequence is that traffic will stabilize at about 2,000 vehicles per hour at a speed of 30 to 40 mph (48 to 64 km/h), as shown by the curved solid line on figure (4.2.3-1). Often,

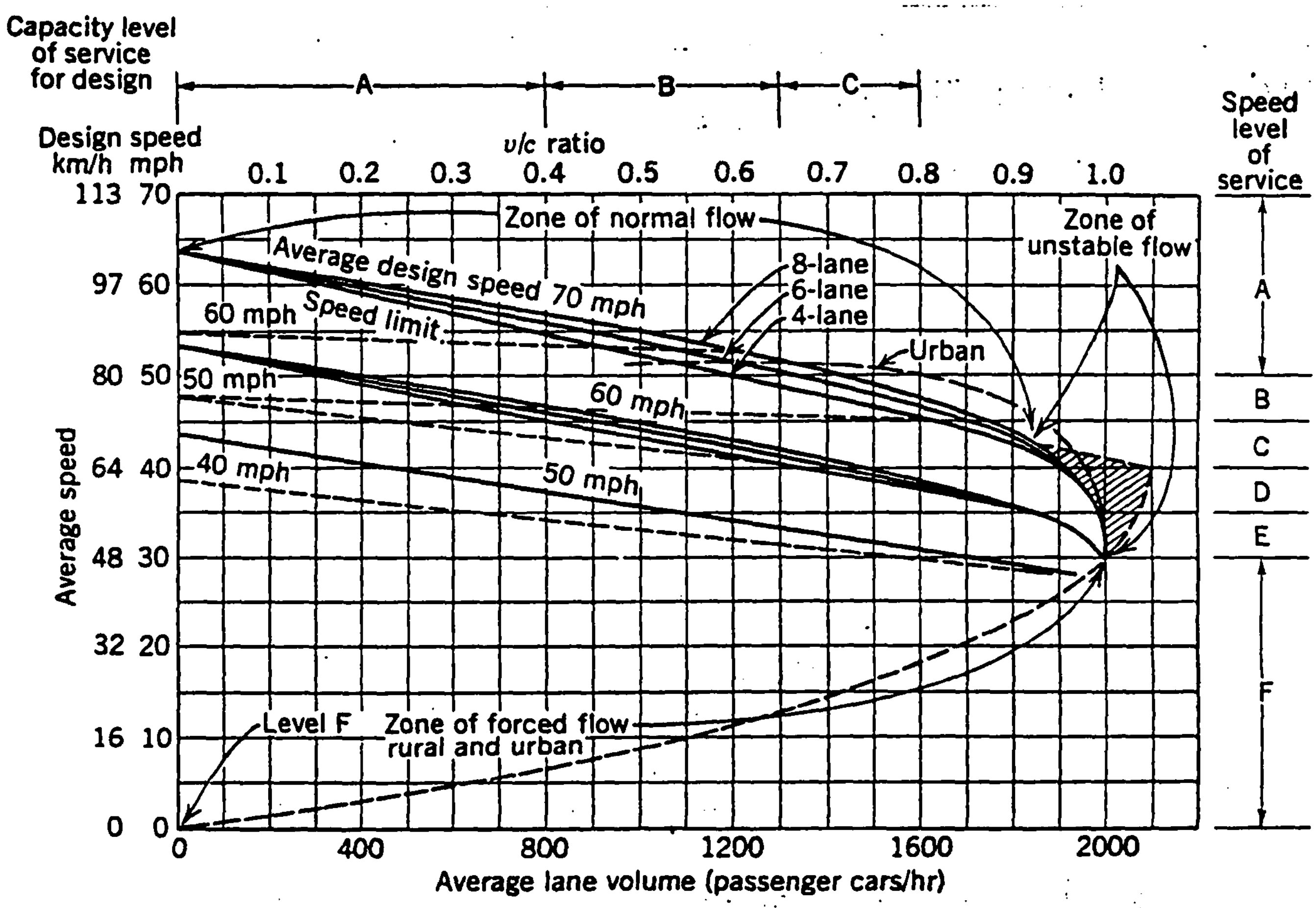
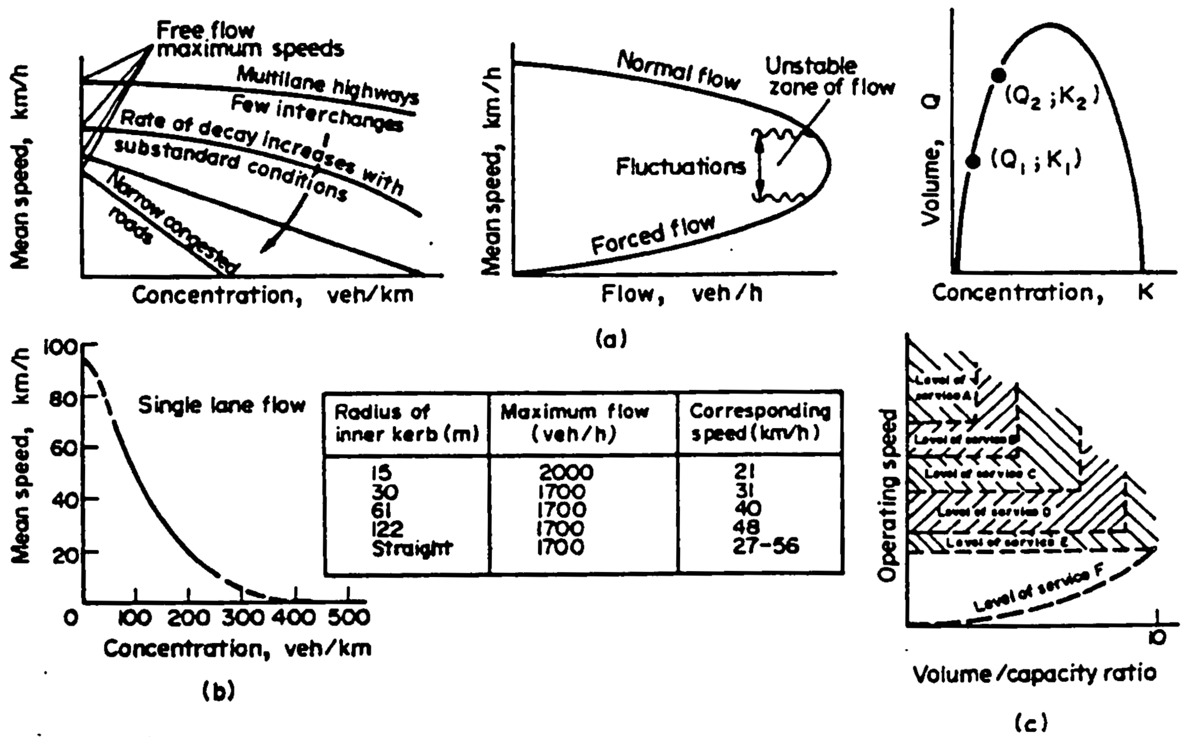


Figure 4.2.3-1 Typical relationships between flow volumes per lane and average speed in one direction of travel under ideal uninterrupted flow conditions on freeways and expressways.
(Reproduced from reference No.110)

however, the quality of flow deteriorates and a substantial drop in speed occurs; in extreme cases vehicles may come to a full stop. In this case the volume of flow quickly decreases as traffic proceeds under a condition known as "forced flow". Volumes under forced flow are shown by the shaded curve at the bottom of figure (4.2.3-1). Reading from that curve, it can be seen that if the speed falls to 20 mph, the rate of flow will drop to 1,700 vehicles per hour; at 10 mph the flow rate is only 1,000; and of course, if vehicles come to a complete stop, the rate of flow is zero. The result of this reduction in flow rate is that following vehicles must all slow down or stop and the rate of flow falls to the levels shown. Even in those cases where the congestion lasts for a few seconds, additional vehicles are affected after the congestion at the original location had disappeared. A "shock wave" develops which moves along the traffic lane in the direction opposite to that of vehicle travel. Such waves have been observed several miles from the scene of the original point of congestion.

Congestion is caused by the flow demands of arrivals in a system requiring a service which has a restriction on its availability, and by irregularities in either the demand or service operation, or in both. This is a queueing system and traffic can be defined as queueing when a following driver must immediately react to a speed reuction made by a preceding vehicle (105). Figure (4.2.3-2) shows the general relation-



Radius of inner kerb (m)	Maximum flow (veh/h)	Corresponding speed (km/h)
15	2000	21
30	1700	31
61	1700	40
122	1700	48
Straight	1700	27-56

Figure 4.2.3.-2 (a) Diagrammatic speed, concentration and flow relationships.

(b) Experimental speed concentration values.

(c) Concept of levels of service at operating speed and volume/capacity ratio.

(Reproduced from reference No.109)

ships between flow, speed and concentration. The relationship between concentration and speed as shown in figure (4.2.3-2)was determined in single lane conditions, mainly on a test track, by the Transport and Road Research Laboratory (105).

The speed-volume relationships for multilane highways may be summarized as follows ;

> Beginning with the appropriate design speed curve as shown in figure (4.2.3-1), progress from the upper left to the right down the slope of the speed-volume curve. At the nose, the flow may stabilize; on the other hand it may break down. In this case speed-volume relationships follow the dashed curve downward to the left to the origin. This progression passes through the zones of normal flow, unstable flow where actual behaviour of traffic is highly unpredictable, and possibly into forced flow where volume is substantially reduced.

Effects of the imposition of speed limits of 60, 50 and 40 mph (97, 80 and 64 km/h) are suggested by the dotted lines on figure (4.2.3-1).

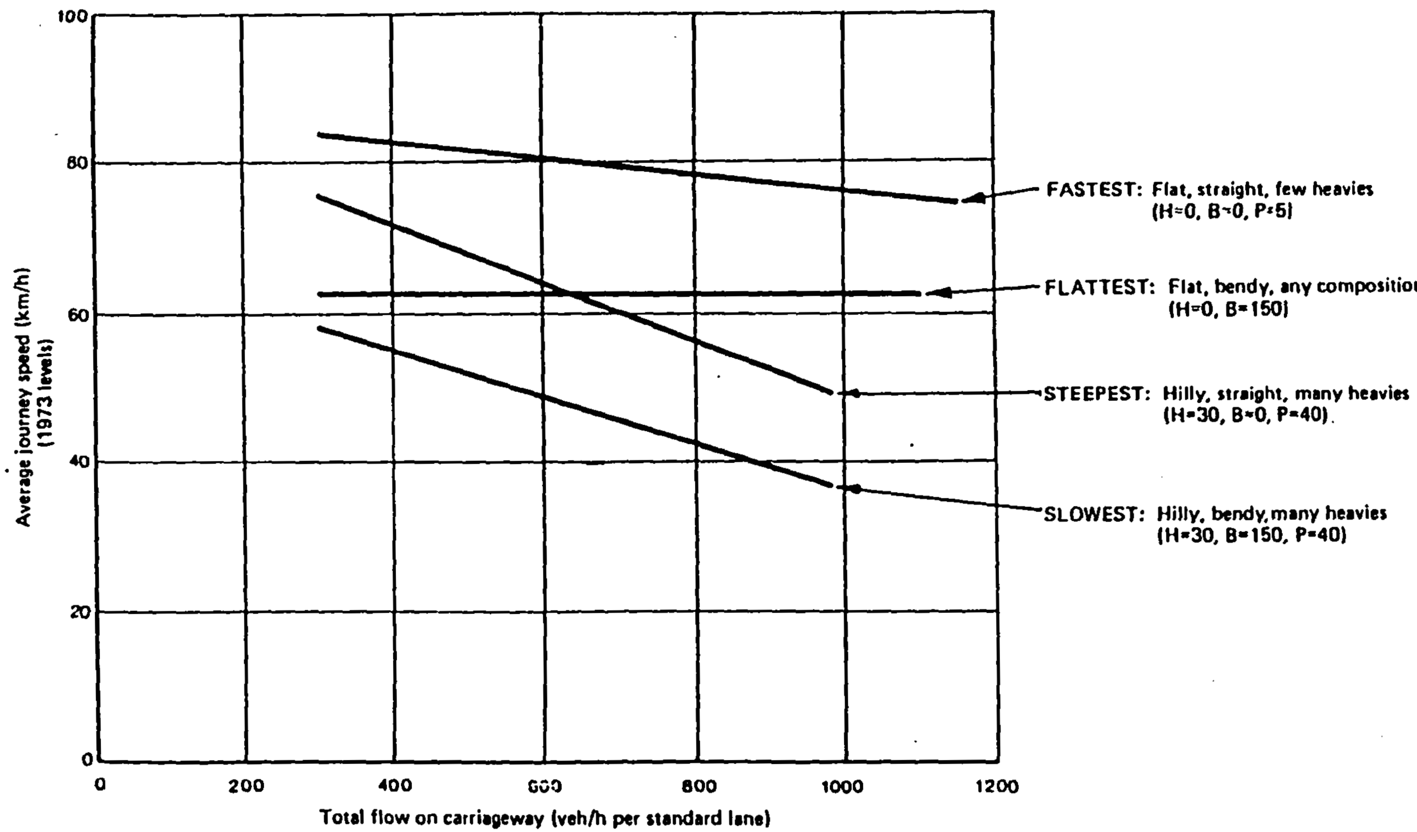


Figure 4.2.3 - 3 Variation of speed/flow relation on single two lane rural road.
(Reproduced from reference No.113)

4.2.4 - Speed-flow relationships for two-lane highways

Figure (4.2.4-1) shows speed-volume relationships for two-lane, two-directional roadways (106). At low volumes where there is little interference from other vehicles, drivers proceed at their desired free-flow speeds. But as volumes increase, interference between vehicles causes speed to fall as shown in figure (4.2.4-1), and as with a lane on a multilane facility, if serious congestion develops, speed falls sharply and the forced flow condition develops as shown by the broken line at the bottom of the graph.

Figure (4.2.4-1) shows that the capacity for a two-lane highway totals 2000 vehicles, which is only half that of two lanes in the same direction. This can be explained as all the flow is in one direction on a two-lane road and vehicles can keep one lane filled by immediately passing into the gaps that form. This single lane can then carry the same number of vehicles as a lane of a multilane facility. However, when passing is restricted by vehicles from the opposite direction, the spaces between vehicles cannot be filled. Instead, queues of vehicles form in each direction behind slower-moving vehicles until the spaces between queues become long enough to permit passing. After a period during which passing is possible, new queues form. As already mentioned, total capacity for both directions is 2,000 vehicles per hour. This can be 2,000 vehicles in one direction with

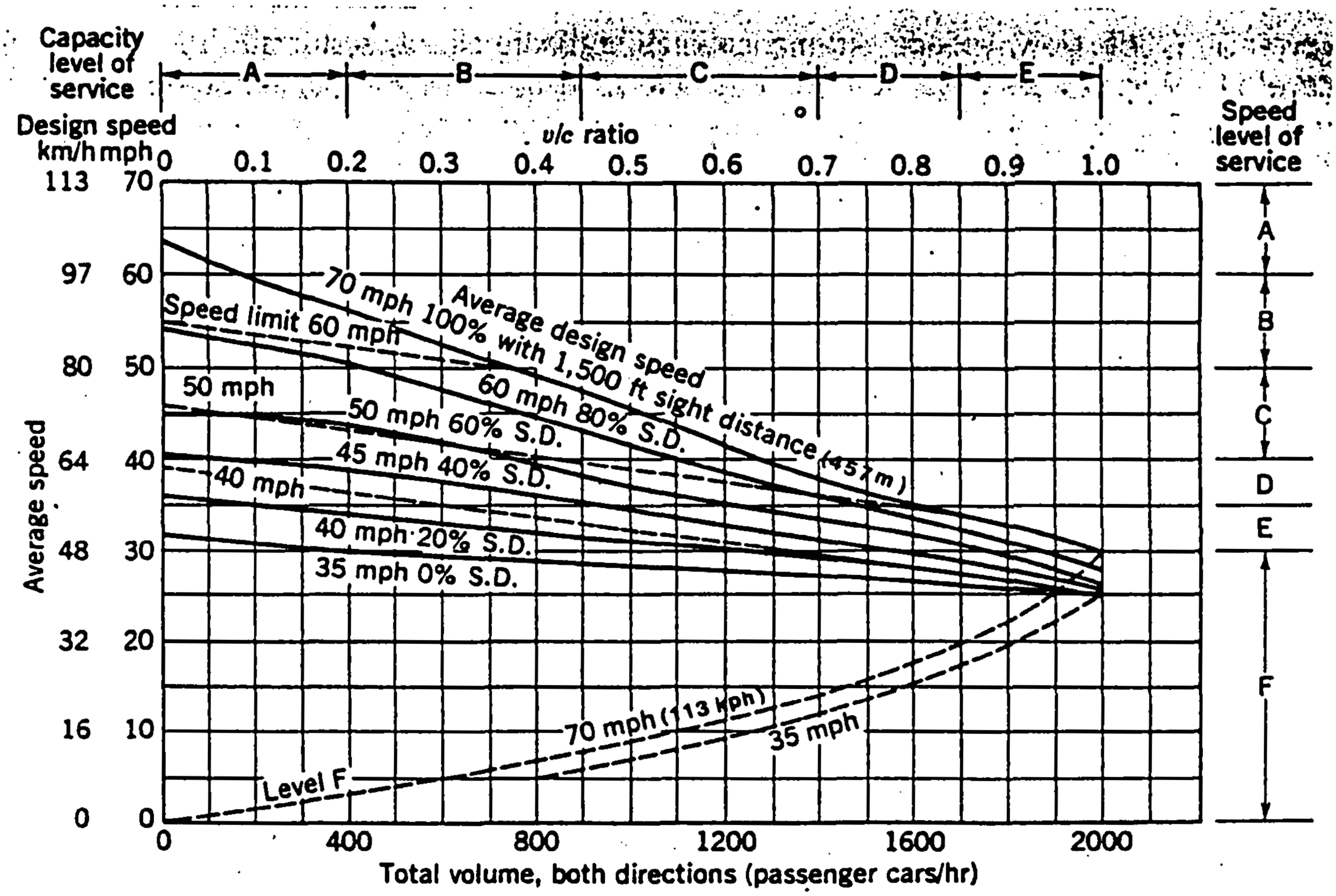


Figure 4.2.4-1 Typical relationships between total volume for both directions of travel and average speed under ideal uninterrupted flow conditions on two-lane highways. (Reproduced from reference No.117)

none from the other, 1,000 from each, or any other combination adding to 2,000 vehicles.

With three-lane roads the centre lane is used for passing in either direction. Under ideal conditions, vehicles can completely fill both inside lanes by passing in the centre one. Thus, capacity can reach a total of 4,000 vehicles for both directions. In the United States, capacity of three-lane roads is only of concern for existing highways since new ones are seldom built because of poor accident experience.

The earliest investigations of stream flow were concerned with speed-density relationship. Greenshields (107) in an early investigation of traffic characteristics, proposed a linear relationship between speed and density in his 1934 study of capacity, a model which has the advantage of simplicity, and which provides a good fit to observed data in many cases.

The mathematical expression of Greenshield's model is:

$$S = S_f \, (1 - D/D_j)$$

where

S = speed

D = density, in vphpl

S_f = free-flow speed

D_j = jam density, in vphpl.

Some observers have suggested a model based on a "one-dimensional" fluid state resulting in the following non-linear relationship (103);

$$S = Sc\ c\ \ln\ (D_{j/D})$$

where Sc equals the critical speed (at capacity).

This model is useful in describing congested flow but it breaks down at low densities, as the theoretical speed at zero density approaches infinity.

Aerde and Yagar (108) investigated the effects of traffic volume on speeds of two-lane rural highways using a large bank compiled in Ontaria, Canada in 1980. These effects were examined for the entire practical range of volumes using two types of linear models. The first considers volumes of cars, trucks, recreational vehicles and other vehicles in the direction being analyzed, and the total volume in the opposing direction. The second model considers only aggregated passenger car units in the main direction and opposing direction. The 10th, 50th and 90th percentile speeds are estimated using each of the above models, so that the ultimate decision-maker can cater to any chosen sub-population or estimate the fraction of drivers that drive at any given speed. It was found that speeds are generally quite insensitive to volumes for a large practical range of volumes and the percentile speed curves tend to converge as main directional volumes increase. More recent work aimed at predicting the effects of volumes on vehicle speeds has been conducted by O'Flaherty and Coombe (109), the Transport and Road Research Laboratory (111),Forsgate and Hammond (112) and Duncan (113).

The Transport and Road Research Laboratory produced a set of speed/flow formulae for rural roads based on 1968 data. The average two-lane highway with a lane width of 3.65m, had a speed for zero traffic volume of 66 km/h. Its speed reduction coefficient was 10 km/h/1,000 vehicles for an average composition of 15% heavy vehicles. This results in a speed/volume curve of the form: speed = 66 - 10 x vol. Duncan (104) produced linear speed prediction models as a function of light and heavy vehicles in the main and opposing directions. A speed reduction of 11 km/h was estimated for an increase in traffic flow of 1,000 veh./h in the main direction, assuming a composition of 15% heavy vehicles. Duncan's investigation (1974) (104) of rural speed/flow relations were based on previous work carried out by the Transport and Road Research Laboratory and were published in the Laboratory's Leaflet No. LF170, which was then replaced by Duncan's study published in the Transport and Road Research Laboratory No. LR651. The following speed formulae were used for two-lane roads with an average carriageway width of 6.70m or 1.83m standard lanes,

$$V_L = 86.5-16.7x\frac{550}{1000} - 12.8x\frac{H}{100} - 14.5x\frac{B}{100} \quad \ldots \quad (1)$$

and

$$V_H = 69.5- 4.1x\frac{550}{1000} - 19.3x\frac{H}{100} - 8.6x\frac{B}{100} \quad \ldots \quad (2)$$

Equations (1) and (2) were combined to give the average speed (with P per cent of heavy vehicles) as;-

$$V_o = 77.3 - \frac{P}{10.0} - \frac{H}{7.25}\left(\frac{196+P}{211}\right) - \frac{B}{7.35}\left(\frac{245-P}{230}\right) \quad \ldots \quad (3)$$

Equation (3) then modified to take into account free-speed condition and average traffic composition;

$$V_o = 75-\left(\frac{P-15}{10}\right) - \frac{H}{7.5}\left(\frac{185+P}{200}\right) - \frac{B}{7.5}\left(\frac{215-P}{200}\right)$$

where

V_o is average free-speed

P proportion of heavy vehicles (per cent)

H average hilliness (m/km)

B average bendiness (degree/km)

For low, average and high values of P, and of H and B the above formula gives the following values of free-speed:-

Free-speed	P=5			P=15			P=40		
(km/h)	H=o	H=15	H=30	H=o	H=15	H=30	H=o	H=15	H=30
B = O	76.0	74.1	72.2	75	73	71	72.5	70.3	68.0
B = 75	65.5	63.6	61.7	65	63	61	63.7	61.5	59.3
B = 150	55.0	53.1	51.2	55	53	51	55.0	52.7	50.5

Figure (4.2.3-3) shows the variation in speed/flow relationship as a result of Duncan's investigations.

4.3 - Capacity studies

4.3.1 - Introduction

For the purpose of the study, 'highway capacity' is as defined in the Highway Capacity Manual 1985 (103):

> 'the capacity of a facility is defined as the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions'.

Capacity in Great Britain is defined in terms of standard and maximum working levels of hourly flow, recognising the differences in expectations of drivers in rural and urban situations. Drivers expect fewer overtaking opportunities and a slower rate of progression for shorter journey distances in urban areas and practical operating capacities can thus be higher. Design year demand for rural roads is based on a primary assessment of the average daily flow (16 hours, 7 days average 0600-2200 hours) in the month of heaviest flow (normally August). Detailed geometric standards are based on a peak hourly flow obtained from the average over 13 weeks of a specified highest hour. The design flows are set down in the Department of the Environment Technical Memorandum H6/74 and H9/76 for rural all-purpose roads and motorways, and for urban roads respectively (118).

4.3.2 - Two-lane capacity prediction

High-volume data on two-lane, two-way rural highways are complex and difficult to obtain. Rarely do such highways operate at volumes approaching capacity and thus the observation of capacity operations in the field is a complex one.

Design flows for both urban and rural highways in the U.K. are given in tables (4.3.2-1) and (4.3.2-2) (116) as laid down in Technical Memorandum H6/74 (118). It should be noted that flows are given in vehicles per hour instead of PCU, but the variable proportions of heavy vehicles are now taken into account. A range of flows is given for design purposes and also for assessing the adequacy of existing roads. Memorandum H6/74 points out that with increasing flows above the standard hourly design levels there will be a progressive decrease in vehicle speeds, decline in driver comfort and corresponding increase in congestion and operational costs. Existing roads can also be assessed for their respective levels of adequacy in relation to the traffic being carried. Flow variations are determined from the ratio of peak hour flow/daily flow (PDR) based on a 16h time period.

Dual carriageway ratios for rural roads are based on the peak hourly, one-way directional flow and the highest monthly average daily flow, while single carriageway peak

Road type	Peak hourly flow veh/hour/carriage-way		16h average daily flow (both directions)		
	Standard	Max. working	Min.	Max. within normal PDR range	Absolute max. a
Rural motorways					
dual 2-lane	2400	3200	35000	45700-48000	56000
dual 3-lane	3600	4800	4500	68200-72000	85000
dual 4-lane	4800	6400	70000	91400-96000	115000
All-purpose dual carriage-way road					
Dual 2-lane	2400	3200	17000	33700-45000	45000
Dual 3-lane	3600	4800	35000	50500-60000	60000
All-purpose single-carriage-way road					
10m wide b	2600	3000	20000	23100-30000	30000 c
10m wide	1900	2300	12000	17700-23750	25000
7.3m wide	1200	1600	2000	12300-15000	17000

a commensurate with exceptionally low values of PDR only
b where centres of interchange are more than 3km apart add 400vph
c only for grade-separated schemes

Table 4.3.2-1 Design flows for motorways and rural all-purpose roads when heavy vehicles comprise < 15 per cent of flow.
(Reproduced from reference No.116)

	Peak hourly flows veh/hour/both directions	Peak hourly flows veh/hour/carriage-way	
Road type	single 2-lane 7.3m wide	dual 2-lane	dual 3-lane
Urban motorways		3600	5700
All-purpose roads, no frontage access, no standing vehicles, negligible cross-traffic	2000	3200	4800

Table 4.3.2-2 Design flows for urban roads when heavy vehicles comprise $\ngtr$ 15 per cent of flow. (Reproduced from reference No.116)

hourly values are combined for both directions of travel (table 4.3.2-1).

Lane capacities vary according to the geometric layout and traffic operating conditions. The highest category types of urban motorway with grade separation and infrequent access have practical operating capacities of 1,500 passenger car units (PCU's) per lane per hour. All purpose routes without frontage access and with full no-waiting controls have capacities of about 1,200 PCU per lane per hour, but this drops to 800 PCU per lane per hour with increasing frequency of major junctions. On other routes the capacity is more often restricted to that of the "bottleneck" junctions.

The latest Highway Capacity Manual (Special Report No. 209, 1985) gives a sample of observed high volume on two-lane rural highway (table 4.3.2-3) and the manual emphasized that the values given in table (4.3.2-3) are not to be taken to represent absolute capacity for the specified facility. The values given are for good operating conditions.

European observations on two-lane, two-way rural highways have been reported at far higher volumes than those given by Highway Capacity Manual (table 4.3.2-3). Volumes of more than 2,700 vph have been reported in Denmark, 2,800

LOCATION	TOTAL VOLUME (VPH)	PEAK DIR. VOLUME (VPH)	OFF-PEAK DIR. VOLUME (VPH)
	2-LANE HIGHWAYS		
U.S. 50, Lake Tahoe, California	1690	1140	550
U.S. 77, Fremont, Nebraska	862	—	—
Hwy. 1, Banff, Alberta, Canada	2000	—	—
Hwy. 1, Banff, Alberta, Canada	2450	—	—
Hwy. 35, Kirby, Ontario, Canada	2050	—	—
Hwy. 35/115, Kirby, Ontario, Canada	2250	—	—
Calif. 84, Fremont, California	2364	1825	539
Trans-Canada Hwy., Alberta, Canada	1391	1150	241
Wasatch Blvd., Salt Lake City, Utah	2198	1504	694
	2-LANE BRIDGES/TUNNELS		
Sagamore Bridge, Hudson, New Hampshire	2180	—	—
Underwood Bridge, Hampton, New Hampshire	1932	1061	871
Stanley Viaduct, Decatur, Illinois	1919	971	948

Table 4.3.2-3 Maximum Observed Hourly Volumes on Two-Lane Rural Highways.
(Reproduced from reference No.103)

vph in France, 3,000 vph in Japan and 2,450 vph in Norway (114). Some of these volumes have contained significant numbers of heavy goods vehicles, some as high as 30 per cent of the traffic stream.

The difficulty experienced in the United States in observing capacity operations on two-lane highways was the problem in terms of suggesting a standard value for use in computational procedures. The procedures for such highways are based on a combination of field observation and simulation, which suggest that a maximum capacity of 2,800 pcu/h be adopted as a total in both directions under ideal conditions. These ideal conditions include a 50/50 directional distribution of traffic.

4.4 - Level of services concept

As discussed in previous sections of this chapter, operating speed decreases and driver restrictions become greater as traffic volume increases. The concept of level of service has replaced (practical) capacity. Level of service is associated with the different operating conditions which occur on a facility when it accommodates various traffic volumes. It is a qualitative measure of the effect of a number of factors, which include:

i) Speed and travel time
ii) Traffic interruptions
iii) Freedom to manoeuvre
iv) Driver comfort and convenience
v) Safety
vi) Vehicle-operating costs

The concept of level of service is carried out throughout the Highway Capacity Manual (1965 and 1985) and it is applied to all highway elements. Six levels of service have been established, designated by the letters A through F, providing for the best to worst service in terms of driver satisfaction. For a given highway facility, different levels of service will be selected to provide for appropriate operating characteristics on the various components of the facility. However, these operating conditions should be in harmony with each other; that is, they should be of approximately equal acceptability to average drivers. Different highway elements and types of facilities include: intersection, ramp, weaving section, ramp terminal, speed-change lane, freeway, uncontrolled-access multilane highway, two-lane or three-lane highways, arterial street, etc. There is an important distinction between capacity and level of service. A given lane or roadway may provide a wide range of level of service (depending essentially on speed and volume), but the lane or roadway has only one capacity. In practice, any given highway, or component, may operate at a wide range of levels of service, depending upon the time of day, day of week, period of the year and traffic composition.

Service volume is the maximum number of vehicles that can pass over a given section of a lane or roadway, in one direction on multilane highways (or in both directions on a two or three-lane highway),during a specified time period, while operating conditions are maintained corresponding to the selected or specified level of service.

Freeway operating characteristics include a wide range of rates of flow over which speed is relatively constant (103). This means that speed alone is not adequate as a performance measure by which to define levels of service. Although speed is a major concern of drivers with respect to service quality, freedom to manoeuvre and proximity to other vehicles are equally important parameters. These other qualities are directly related to density of the freeway traffic stream. Further, rate and flow increases with increasing density throughout the full range of stable flows. For these reasons, density is the parameter used to define levels of service for basic freeway segments. The densities used by the new Highway Capacity Manual (Special Report 209) to define the various levels of service (LOS) are as follows:

Level of Service	Density (pc/mi/ln)
A	12
B	20
C	30
D	42
E	67

These values are boundary conditions representing the maximum allowable densities for the associated level of service. The LOS-E boundary of 67 pc/mi/ln (67 passenger car/mile/lane), has been generally found to be the critical density at which capacity most often occurs. Level-of-service criteria for basic freeway segments are given in table (4.4. -1) for 70 mph (113 km/h), 60 mph (97 km/h) and 50 mph (80 km/h) design speed elements. To be within a

Level of Service	Density (PC/MI/LN)	70 MPH (113KPH) Design Speed			60 MPH (97KPH) Design Speed			50 MPH (80KPH) Design Speed		
		Speed[a] (MPH)	v/c	MSF[b] (PCPHPL)	Speed[a] (MPH)	v/c	MSF[b] (PCPHPL)	Speed[a] (MPH)	v/c	MSF[a] (PCPHPL)
A	$\leq$ 12	$\geq$ 60	0.35	700	-	-	-	-	-	-
B	$\leq$ 20	$\geq$ 57	0.54	1,100	$\geq$ 50	0.49	1,000	-	-	-
C	$\leq$ 30	$\geq$ 54	0.77	1,550	$\geq$ 47	0.69	1,400	$\geq$ 43	0.67	1,300
D	$\leq$ 42	$\geq$ 46	0.93	1,850	$\geq$ 42	0.84	1,700	$\geq$ 40	0.83	1,600
E	$\leq$ 67	$\geq$ 30	1.00	2,000	$\geq$ 30	1.00	2,000	$\geq$ 28	1.00	1,900
F	> 67	< 30	c	c	< 30	c	c	< 28	c	c

Table 4.4-1 Levels of Service for Basic Freeway Sections (Reproduced from reference No.103)

a. Average travel speed.

b. Maximum service flow rate per lane under ideal conditions.

c. Highly variable, unstable.

d. One PCPMPL = 0.625 PCPKPL.

Note: All values of MSF rounded to the nearest 50 pcph.

given level of service, the density criteria must be met. The average travel speeds and the maximum service flow rates indicated in the table are under ideal conditions for the given densities.

Figure (4.4-1), shows the relationship between flow densities, speed and different levels of service. Figure (4.4-1), shows in two different forms, the relationships of some of the better-understood measures of quality of flow to the volume/capacity ratio and to the various levels of service for uninterrupted flow conditions (figure 4.4-1(a)). As shown in figure (4.4-1(b)), when operating speed is in the highest range, density is at its lowest value.

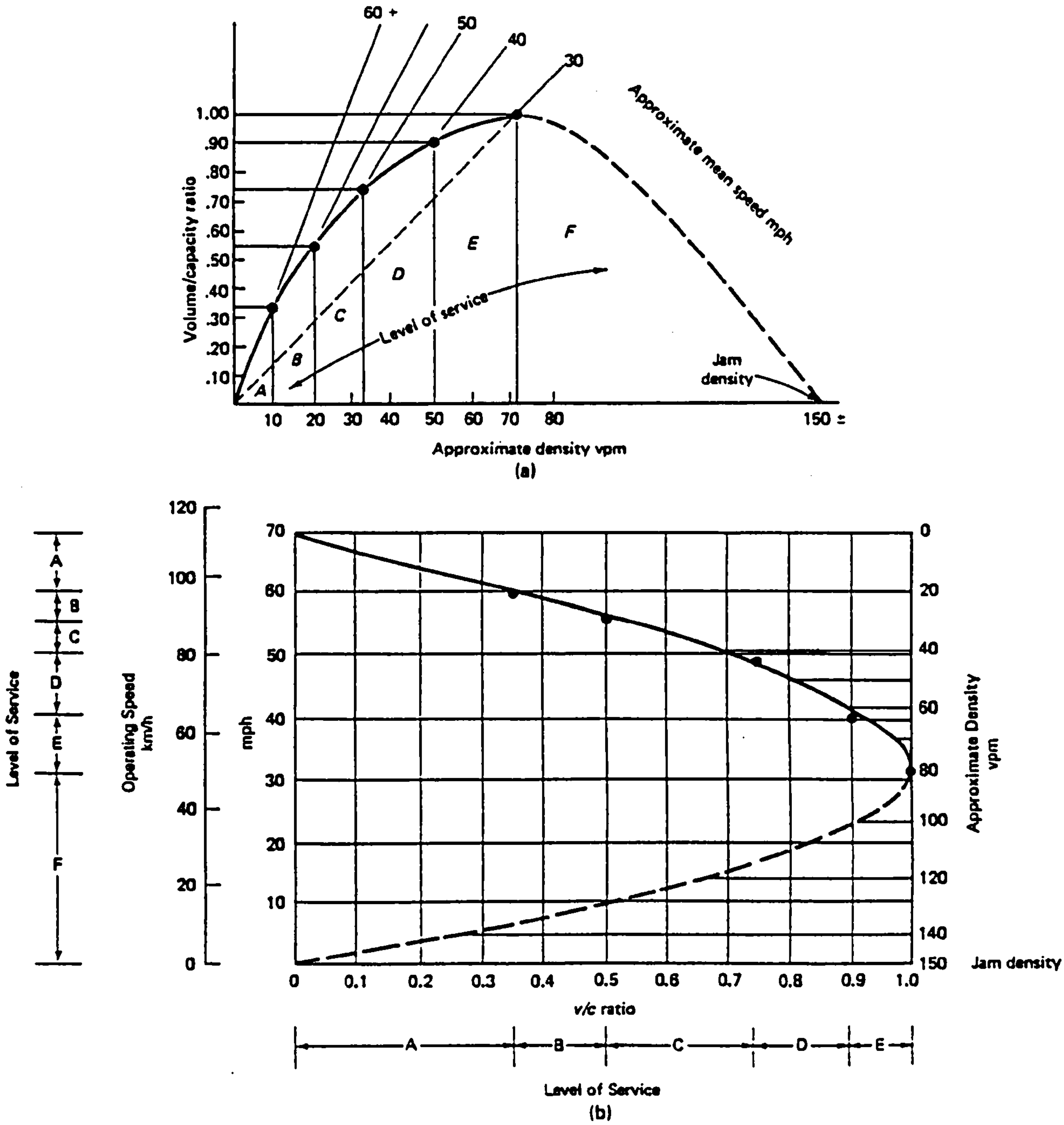


Figure 4.4-1 (a) Relationship of levels of service to some measures of quality under uninterrupted flow conditions (basic form).
(b) Relationship of level of service to some measures of quality of flow under ideal uninterrupted flow conditions (approximate form, based on familiar speed-volume curve).
(Reproduced from reference No.117)

4.5 - Theory and study approach

4.5.1 - Introduction

This part of the study will investigate the significant effects of various vehicle types on rural highway traffic flow, on two and three-lane motorways. The investigation falls into four sections:

(a) the study of vehicle type speed characteristics, which are the criteria used later in the investigation of vehicle type effects on other highway traffic parameters;

(b) the study of traffic composition and vehicle type distribution per lane of travel;

(c) introducing a procedure for computing passenger car unit equivalencies for two-lane motorways;

(d) the study of vehicle type effect on overtaking procedures, speed/flow relationships and capacity.

4.5.2 - Vehicle type speed studies

In this study, vehicle type speed criteria have been investigated, using individual vehicle type performances and their various effects on traffic flow performance on two-lane and three-lane motorways.

Most of the speed measurements made by highway traffic engineers produce a time mean speed because they are obtained by the use of radar speedometers or filming techniques. In the latter, however, the mean travel time (average running speed) is used to calculate the mean speed; then the space mean speed is obtained.

Both radar speedometer and filming techniques were used in this study to obtain speed distribution of individual vehicle type and distribution of speeds per lane of the section of the motorway under investigation.

4.5.3 - <u>Procedure for computing passenger car unit equivalencies</u>

(A) <u>Overtaking method</u>

Passenger car unit equivalencies (PCE's) or adjustment factors for commercial vehicles, buses and recreational vehicles are often required in carrying out highway capacity calculations.

A passenger car equivalent (PCE), in highway capacity analysis, is the number of passenger cars which is roughly the equivalent of one commercial vehicle, bus, or recreational vehicle under prevailing roadway and traffic conditions. The use of such equivalents is central to highway capacity analysis where mixed traffic streams are present and the calibration of these values can have a dramatic impact on capacity analysis computations.

The Highway Capacity Manual (103), discusses the issue of passenger car equivalencies for two-lane and multilane highways and suggests that they can be determined directly by obtaining detailed information on the speeds and headways of different types of vehicles involved during various rates of flow on highways. An average passenger car equivalent is obtained for each vehicle type under each condition. Passenger car equivalents can be calculated with a high degree of accuracy from separate speed distributions of passenger cars and commercial vehicles at any given volume level.

The criterion used by the Highway Capacity Manual* is the relative number of passings that would be performed for a unit length of highway (one kilometer) for uninterrupted flow if each (the passing one and the overtaken one) continued at its normal speed for the conditions under consideration.

Theoretical or desired passing may be defined as the passing by a faster vehicle of a slower vehicle, when each vehicle continues at its normal speed. The theoretical number of passings for passenger car flow may be obtained first by a separation of the total traffic volume into passing and passed vehicles, and then a summation of the total number of overtakings by faster vehicles of slower vehicles can be expressed by:

$$P_p = \sum_{i=1}^{m-1} \sum_{j=2}^{m} C_i \, C_j \left(\frac{1}{U_i} - \frac{1}{U_j}\right),$$

where P_p = the total number of theoretical passenger cars passing within one kilometre, during a given period; U_i, U_j = overall travel speed of slower and faster vehicles, respectively; C_i, C_j = number of vehicles having respective speed, U_i or U_j for given period; i, j = respective index of passed and passing vehicles; m = one-direction passing-

* Trans. Res. A. Vol.14, pp. 241-246, 1980.

car volume for a given period.

The criteria of the number of passings or number of overtakings that would be performed per kilometre of highway forms the basis of passenger car equivalent measurement in this chapter.

A vehicle type factor (Vt) is conventionally used to adjust the flow of mixed vehicles at a rate of Q vehicles/h to the equivalent flow rate of passenger cars only (Q_E).

$$Q_E = Q/Vt$$

The flow-rate per hour (Q) may consist of a mixture of passenger cars, heavy vehicles, buses and coaches. Q is equivalent to a Q_E of 100 per cent passenger cars where the mean speed of passenger cars has been chosen as the measure for equivalence values.

If mean speed of passenger cars is the measure for equivalence, knowing and using these mean speeds is necessary for traffic flows with 100 per cent passenger cars.

Consider a section of arterial two-lane. one-way highway and assume that a stream of slower-moving vehicles (Q_1); with speed (S_1)m is overtaken by a faster travelling vehicle with speed (S_2) in flow (Q_2). How many acts of overtaking will take place per one kilometer travelled by the faster-moving vehicle, see figure (4.5.3-1)?

From a fundamental law of traffic;

Flow = Speed x Density

$$Q = S \times D$$

for slower moving stream

$$Q_1 = S_1 \times D_1$$

$$\frac{1}{D_1} = \frac{S_1}{Q_1} = \text{spacing of vehicles}$$

relative speed of the faster vehicle = $(S_2 - S_1)$

where S_2 = speed of the faster-moving vehicle.

Average time interval to overtake a slow moving vehicle

$$= \frac{\text{distance travelled}}{\text{relative speed}} = \frac{S_1}{Q_1(S_2-S_1)} \quad \cdots\cdots \quad (4.5.3\text{-}1)$$

time taken for faster vehicle to travel one kilometer

$$= \frac{1}{S_2} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (4.5.3\text{-}2)$$

Number of overtakings per one kilometer travelled by faster vehicle = (2)/(1)

$$= \frac{Q_1(S_2-S_1)}{S_2 S_1} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (4.5.3\text{-}3)$$

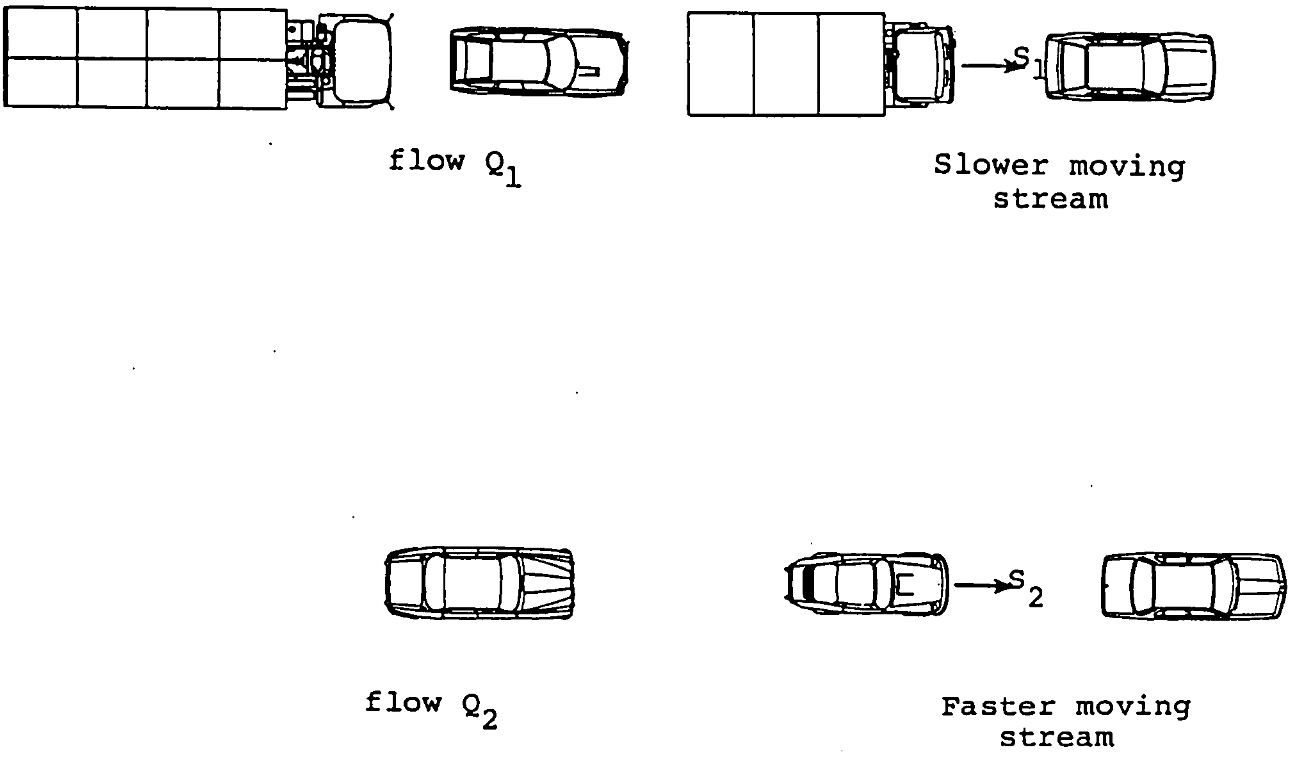


Figure 4.5.3-1 Overtaking procedure between faster vehicles in stream flow Q_2 and slower vehicles in stream flow Q_1.

For a flow Q_2 of faster-moving vehicles, total number of overtakings;

$$N = \sum_{i=1}^{n} \sum_{j=2}^{m} Q_i \, Q_j \frac{(S_j - S_i)}{S_i S_j} \quad \ldots\ldots\ldots(4.5.3\text{-}4)$$

where N = the sum of overtakings

Q_i, Q_j = number of slower and faster vehicles respectively

S_i, S_j = speed of slower and faster vehicles respectively, within one kilometer of the highway.

n = no. of classes of slower moving vehicles

m = no. of classes of faster moving vehicles

Equation (4.5.3-4) permits computations for any particular distribution for any slower vehicle to arrive at a PCE.

The calculation of passenger car unit equivalencies of faster-moving vehicles (100 per cent passenger cars) is given by;

$$PCE = \frac{N/\text{commercial/h}}{N/100 \text{ passenger cars/h}} \quad \ldots.(4.5.3\text{-}5)$$

The PCE valu for commercial vehicles obtained from the above equation (No.5), is the passenger car equivalent of a commercial vehicle travelling at a certain speed class.

By using equations (4.5.3-4) and (4.5.3-5), passenger car equivalent values can be obtained using speed distributions for various levels of service and PCE values versus speed curve can be produced.

The PCE values for different levels of service can be derived if the overall travel speed distribution of passenger cars is known for each level of service. For upgrade or downgrade, the calculation is based on the length and steepness of the grade.

(B) Headway method

The headway method is another approach to passenger-car equivalent determination and is best suited to determine commercial vehicle equivalencies on level highways (103). The method is based on the concept that a heavy commercial vehicle occupies more space than a single passenger car and travels at a lower speed and therefore reduces capacity. The procedure involves the measurement of the interval (headway) between vehicles and their speed. This procedure does not consider passing (overtaking) as does the previous method. The basic equation for the headway method is as follows:

$$E = (h/p - c)/t$$

where E = PCE for heavy vehicle

h = average headway for a sample of cars and heavy vehicles

p = average headway for all passenger car sample

c = proportion of cars

t = proportion of heavy vehicles.

4.5.4 - An analysis of vehicle type effect on highway traffic parameters

Vehicle-type properties, which include power-to-weight ratios (acceleration capabilities), overall lengths, driver-eye heights and braking capabilities are the most important factors in measuring vehicle-type performance, especially of heavy commercial vehicles, and their influences on highway traffic parameters. The influences of heavy commercial vehicles including speeds, sight distances, passing sight distances, stopping sight-distances for crest vertical curves, lane-changing and over-taking procedure have an important effect on highway capacity.

This study is an attempt to investigate vehicle type effect on highway capacity by investigating vehicle type speeds and overtaking procedures, and speed/flow/vehicle type relationships for two-lane traffic flow.

Vehicles are analysed according to lane location, type and time period; and the saturated intervals are considered for the analyses. Speed values for different types of vehicles are analysed by grouping the results to represent values for each type of vehicle using 5 km/h as the class limit. Normal distribution is used for the analysis and to test the goodness of fit a chi-square test was conducted for each set of data.

4.6 - Site sèlection and description

It was not an easy task to locate a stretch of arterial road with dual carriageway that was suitable to film the movement of traffic. This is because the camera had to be positioned so that it would cover a reasonable section of the road (0.6 km approximately) for filming purposes.

A suitable section was located on a stretch of the M621 Motorway. It was found possible to conduct the filming process using a colour video camera (equipment described in Appendix A), by mounting the camera on one of the bridges on the motorway. The bridge forms part of a private road connecting some industrial establishments which are on the opposite side of the motorway to the town of Morley, see figures (4.6-1) and (4.6-2).

This section of the motorway passes through a very busy industrial and commercial area of Leeds and joins the M62 Motorway at junction No. 27.

The section of the highway filmed was very heavily trafficked in both directions, with a high percentage of heavy commercial vehicles and coaches. The section chosen to be filmed was well away from motorway junctions to ensure consistency and uninterrupted flow conditions.

Figure 4.6-1 Camra used for observation.

Figure 4.6-2 Equipment used for observation.

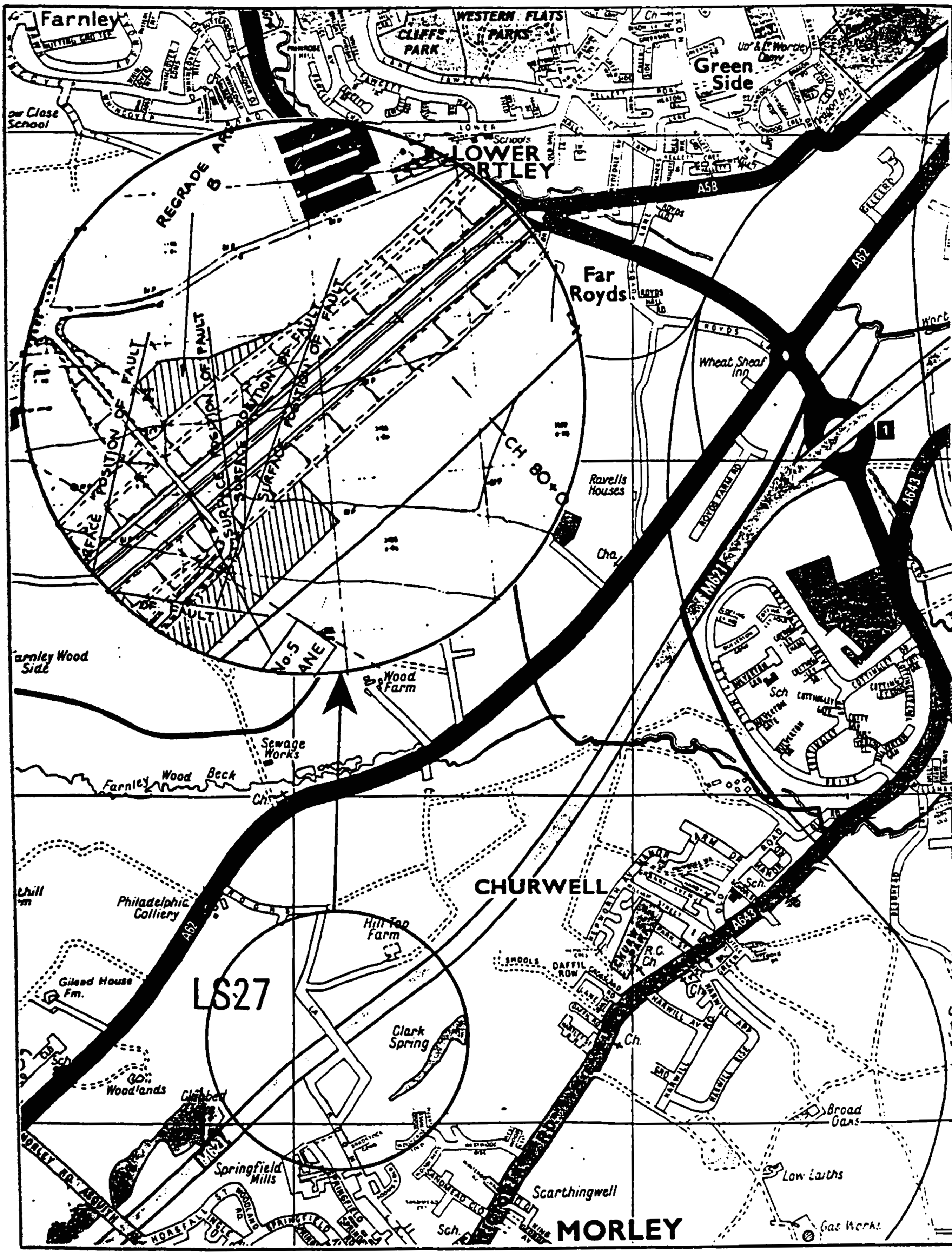


Figure 4.6-3 Section of M621 at Morley.

The M621 Motorway is of two lanes in the direction towards Leeds (East-bound) and three lanes in the direction outward from Leeds (West-bound), towards the M62 Motorway, see figure (4.6-3).

4.7 - Method of observation

4.7.1 - Filming technique and data collection

The site layout for data collection is shown in figure (4.7.1-1) It shows the positions of the video camera A and B. Distances of 30 meters intervals were used in order to construct grid times on the monitor screen for the purpose of film analysis in the laboratory. The distances of 30 meters were measureed and marked on the side of the road with the co-operation of the West Yorkshire Police Authority. The ten marked points were then recorded on a video film by using a flash light pointing at the camera from each marked point, the camera being mounted on the bridge. Ten points were recorded covering a distance of 270 meters which produced ten horizontal grid lines plotted on the monitor screen in the laboratory.

A colour video camera and a radar speedometer (MUNI QUIP DRS-3) were used to analyse speed distribution of different vehicle types in different lanes. The GX-N70E-JVC colour video camera with 8X Auto-Focus zoom lens was used, (as described in Appendix A). The camera was mounted on a bridge to monitor the stretch of the M621 under investigation from positions "A" and "B". see figure (4.7.1-1). The camera was about 6.10m above ground level of the road facing traffic on the three-lane carriageway. It was at position "A" for

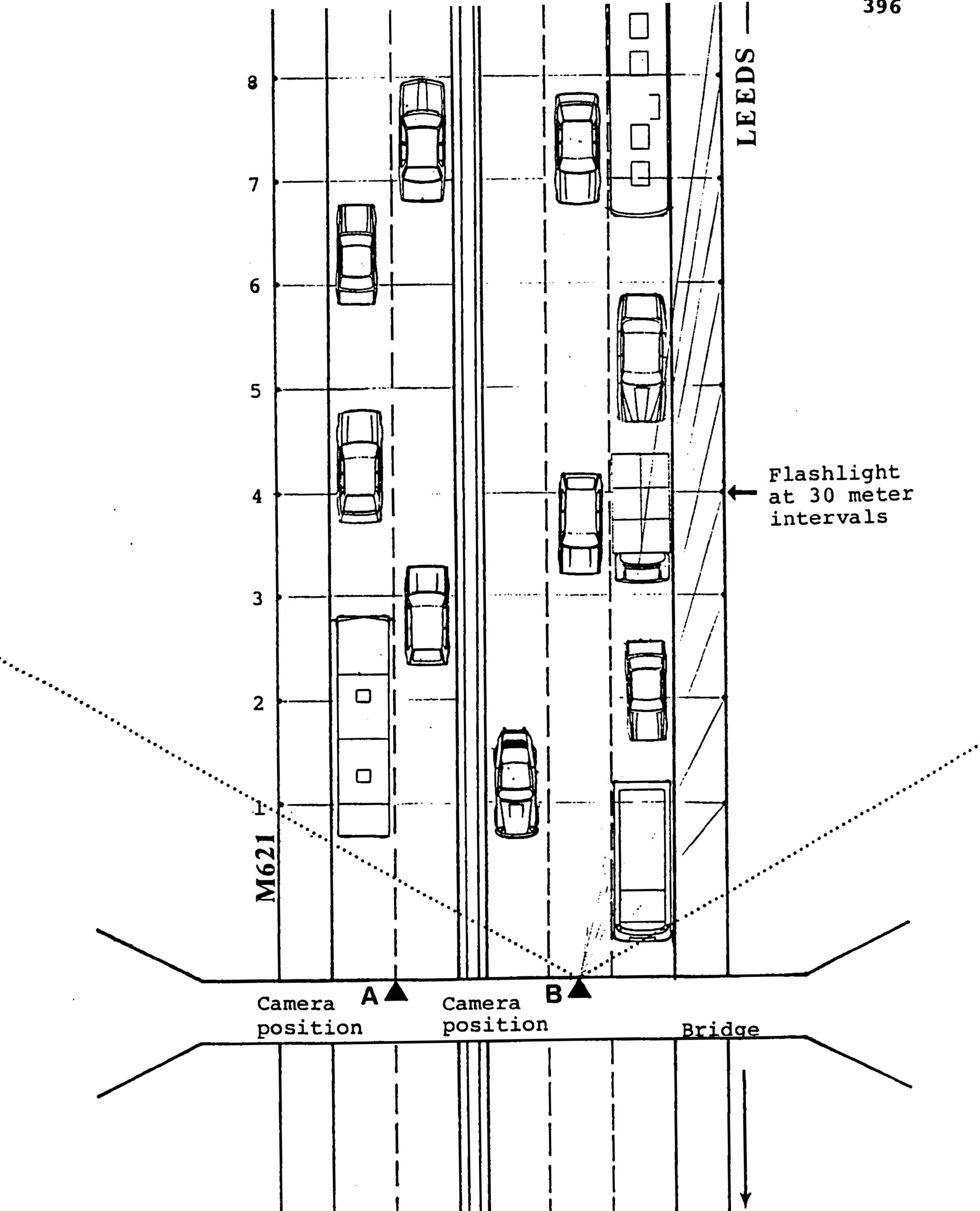


Figure 4.7.1-1 Section of site (M621) filmed, camera positions (A & B) and traffic movements.

traffic travelling towards Leeds and at position "B" for traffic travelling in the opposite direction.

Ten films of 2-hours' duration each were recorded for normal traffic flow periods during the morning hours between 10 a.m. and 12 noon and in the afternoons between 2 p.m. and 4 p.m.

The use of video-recorded tapes enabled each vehicle movement on the section of the road to be studied in detail by examining the movement of vehicles on a frame-by-frame basis, where every 25 frames are equivalent to one second using frame advance facility.

Also the use of video-filming techniques creates a permanent record which provides the opportunity to check the data and thus control their quality at any time.

The radar speedometer Muni Quip DRS-3 was used to measure spot-speeds for vehicle type per lane to study the pattern of speed distribution. The DRS-3 radar is simple to operate and easy to handle and is a crystal-controlled unit which requires no calibration. The read-out accuracy is $\pm \frac{1}{2}$ km/h or mph. The DRS-3 radar has an antenna transmitter which transmits a cone-shaped electromagnetic wave, similar to the beam of a flash light. The radar beam travels at the speed of light in a straight line, until it hits solid objects i.e. vehicles. The part of the reflected beam which returns to the antenna is used to "sense" the speed of a target moving towards or away from the from the radar unit. The DRS-3 radar speedometer can operate at any angle to the line of travel of vehicles. This is because the radar measures the speed at which a target changes its distance from the antenna. A vehicle driving at an angle through the radar beam produces an error reading (cosine error) which is always lower than the actual target speed.

In order to reduce the error in measuring target speed it is advisable that the radar should be aimed from a location as close to the line of travel as practically possible. In highway operations, a single vehicle speed measure situation is ideal. On long and straight roads it may be advantageous to aim the radar at a slight angle. More details about the DRS-3 radar speedometer are given in Appendix A.

4.7.2 - The method adopted for film analysis

Horizontal grid lines were constructed on the monitor screen in the laboratory which was used during film analyses as mentioned in section (4.7.1).

The recorded film of the ten points marked on the on the side of the M621 Motorway was displayed and horizontal lines were drawn at each point on the monitor screen. The distances between the lines on the monitor screen represent the distances between the ten points marked on the side of the road at 30-meter intervals.

When analysing the recorded films in the laboratory on a frame by frame basis, where each 25 frames are equivalent to one second, speed distributions for different types of vehicles per lane were obtained. The accuracy of measuring the time needed for a vehicle to travel between constructed grid lines on the monitor screen is 0.04 seconds, see figures (4.7.2-1) and (4.7.2-2).

Each lane was analysed separately to obtain mean speed distribution for each lane and speed distribution for each vehicle type. Also obtained from analysed film were the speeds of vehicles overtaking slower vehicles over the section of the road filmed. The speed of each vehicle involved in overtaking other vehicles was noted and the number

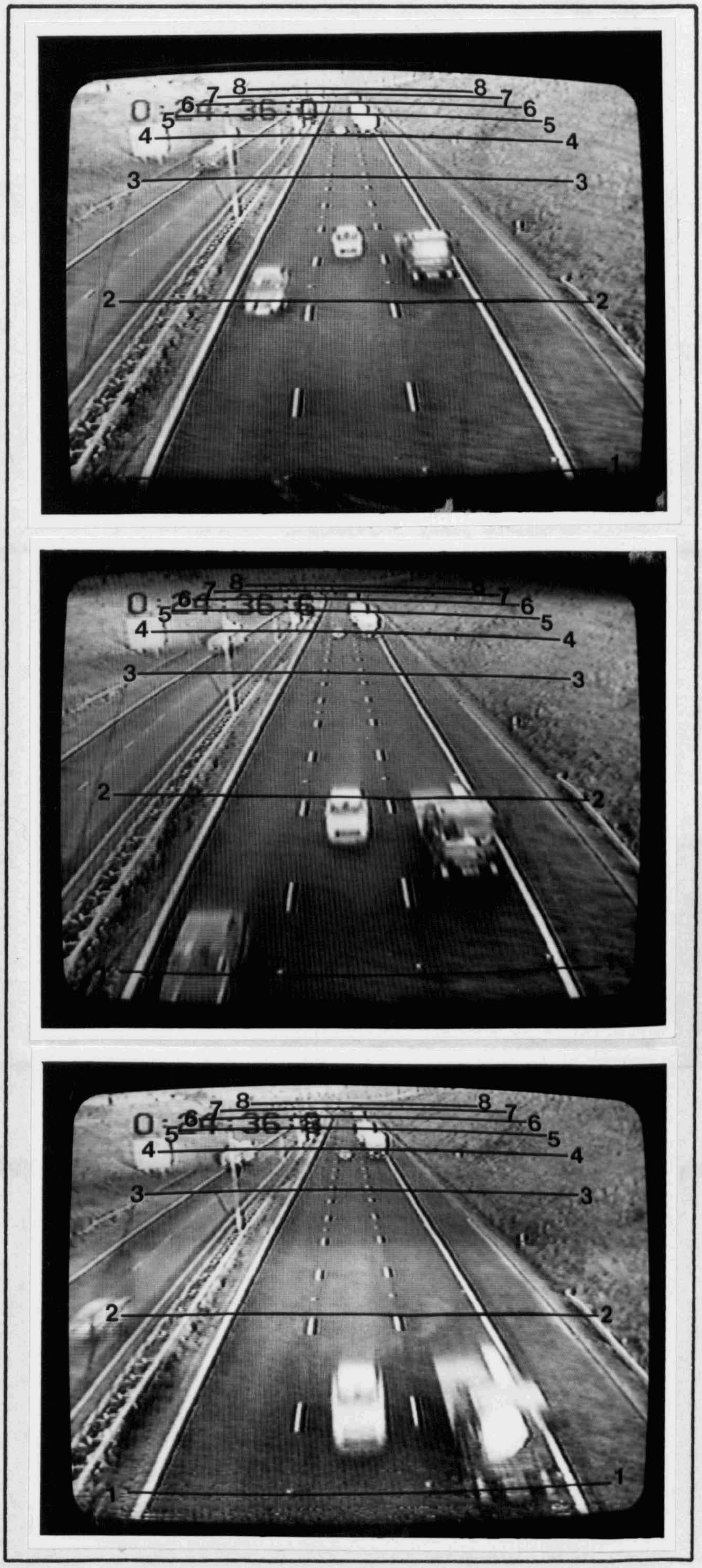


Figure 4.7.2-1 Three frames representing vehicle movement between constructed grid lines on the monitor screen.

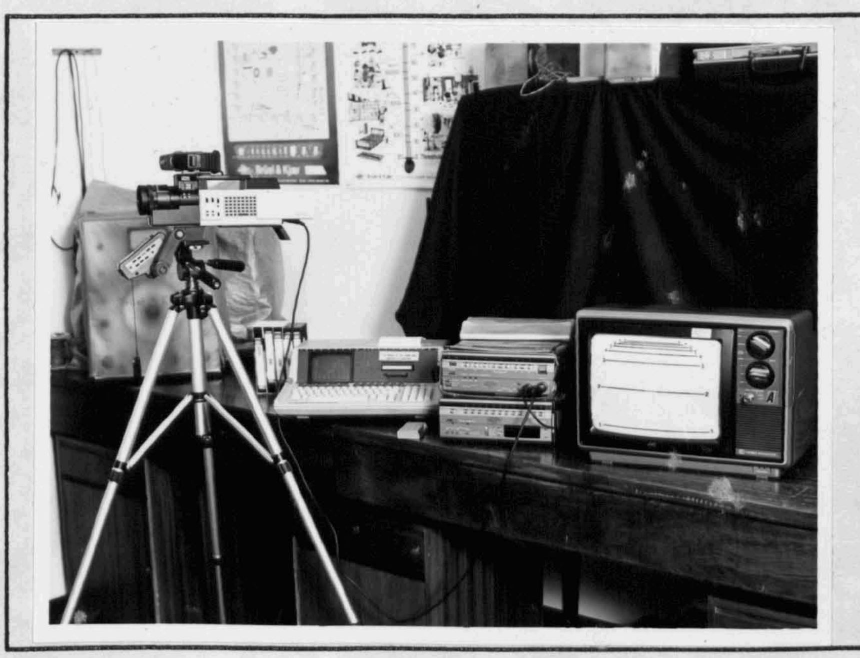


Figure 4.7.2-2 Equipment used for film analysis.

of vehicles that were being overtaken and their speeds were noted.

For example, if a vehicle travelled between grid line number 10 and grid line number 1 and the recorded time was 6.48 seconds,
then;

$$\text{travel speed} = \frac{\text{total distance}}{\text{total time}}$$

total distance = 270 meters = 0.270 km

travel speed = 150 km/hr

Speed distribution of traffic travelling in both directions on the motorway were obtained and recorded on special data form sheets. The data obtained from recorded films was then used to study speed distribution of different vehicle types and their effect on traffic flow.

4.8 - Speed observations analysis

4.8.1 - Vehicle type speed distribution

The characteristics of individual vehicle types travelling along a stretch of road determines their performance and progress along their journey. Drivers' behaviour and therefore, the traffic characteristics of vehicles is generally recognised, under free-flow conditions, to be random in nature.

One of the fundamental parameters for describing vehicle type effects on traffic flow performance is speed distribution. The aim of the investigation was to observe vehicle type speed distribution on two-lane and multilane highways. These observations were obtained from the study site during the morning and afternoon both for peak and off-peak periods. Speed observations were classified as passenger cars, light goods vehicles, heavy goods vehicles and articulated goods vehicles per lane of travel (near side, and far side lanes) for the two-lane motorway and (nearside, middle and far side lanes) for the three-lane motorway.

A speed distribution study was conducted to investigate vehicle type travel patterns and how they are affected by their location in different lanes under free-flow conditions.

4.8.2 - Fitting normal distributions

It is often assumed that speeds are normally distributed. To test this hypothesis, the mean and the standard deviation of the observed speeds were calculated and a normal distribution was fitted to the observed values. The Chi-square goodness of fit test was applied to determine how well speed data agreed with this fit. Tables (4.8.2-1) to (4.8.2-8) show the speeds observed and their distribution for two-lane motorway, and tables (4.8.2-9) to (4.8.2-18) for three-lane motorway.

Each table gives the location of observation, the type of vehicle, time of observation and the results of the Chi-square test. Each table is followed by the normal distribution curve showing the mean and standard deviation. Individual speeds have been grouped into speed classes of 5km/h, an interval which reduces the data into an easily managed number of classes and yet does not hide the form of the speed distribution. For ease in presentation, the columns have been numbered and indicate the following:

Column 1 : Speed class in km/h

Column 2 : Observed frequency in class

Column 3 : Observed frequency of speeds greater than the lower class limit

Column 4 : Column 3 expressed as a percentage

Column 5 : Theoretical frequency of speeds in class, obtained by fitting a normal distribution to the observed values with the same mean and standard deviation

Column 6 : Theoretical frequency of speeds greater than the lower class limit

Column 7 : Column 6 expressed in a percentage.

The results are summarized in tables (4.8.2-9) and (4.8.2-20) which show the mean and Chi-square values for each vehicle type and the observed flow for each lane of the highway.

From these results and with 5 per cent confidence limits, it is concluded that the speeds in all lanes for each vehicle type considered could be regarded as normally distributed.

The results indicate the following;

i - The mean values of the speed of passenger cars travelling in the fast lane are almost the same for peak and off-peak periods. This is because vehicles have to maintain higher speeds while overtaking and to observe the fast-lane overtaking regulation in both periods. These mean values indicate that passenger cars are capable of maintaining

1	2	3	4	5	6	7
45 - 49.9	4	464	100	1.76	462.51	100.00
50 - 54.9	8	460	99.14	5.10	460.75	99.62
55 - 59.9	12	452	97.41	12.30	455.65	98.52
60 - 64.9	21	440	94.82	25.10	443.35	95.86
65 - 69.9	45	419	90.30	42.97	418.25	90.43
70 - 74.9	52	374	80.60	61.90	375.28	81.14
75 - 79.9	66	322	69.40	74.84	313.38	67.76
80 - 84.9	89	256	55.17	74.33	238.54	51.58
85 - 89.9	70	167	35.99	65.38	164.21	35.50
90 - 94.9	50	97	20.91	47.20	98.83	21.37
95 - 99.9	28	47	10.13	28.58	51.63	11.16
100 - 104.9	12	19	4.09	14.30	23.05	4.98
105 - 109.9	5	7	1.51	6.45	8.75	1.89
110 - 114.9	2	2	0.431	2.27	2.30	0.49

Table 4.8.2-1 Light goods vehicles speed distribution (near-side lane/peak periods).

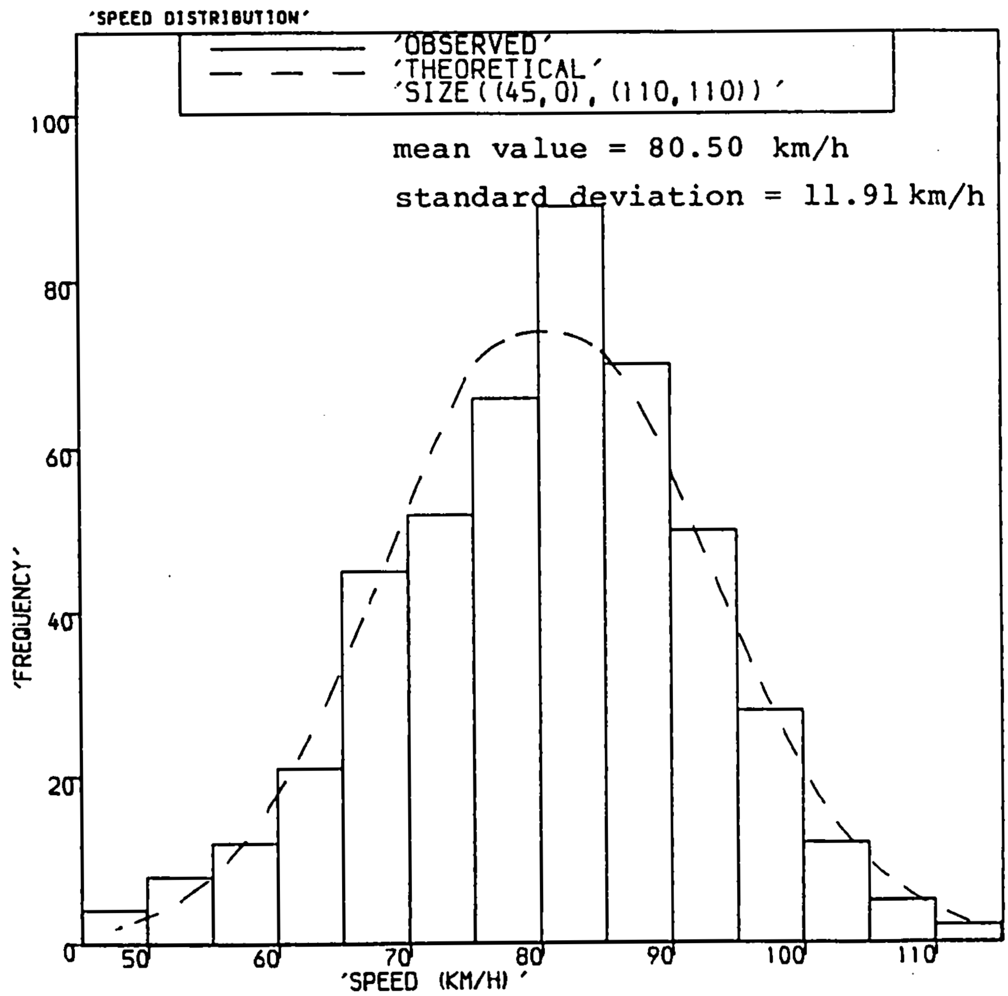


Figure 4.8.2-1 Speed distribution (light goods vehicles).

1	2	3	4	5	6	7
35 - 39.9	2	600	100	4.02	604.06	100.00
40 - 44.9	6	598	99.67	11.76	600.04	99.33
45 - 49.9	38	592	98.67	27.72	588.28	97.39
50 - 54.9	65	554	92.33	59.16	560.56	92.80
55 - 59.9	77	489	81.50	82.8	501.4	83.00
60 - 64.9	94	412	68.67	104.76	418.6	69.30
65 - 69.9	114	318	53.0	105.36	313.84	51.96
70 - 74.9	88	204	34.0	90.36	208.48	34.51
75 - 79.9	54	116	19.33	61.56	118.12	19.55
80 - 84.9	36	62	10.33	34.00	56.56	9.36
85 - 89.9	20	26	4.33	15.30	22.56	3.73
90 - 94.9	5	6	1.0	5.58	7.26	1.20
95 - 99.9	1	1	0.167	1.68	1.68	0.28

Table 4.8.2-2 Heavy goods vehicles speed distribution (near-side lane/peak periods).

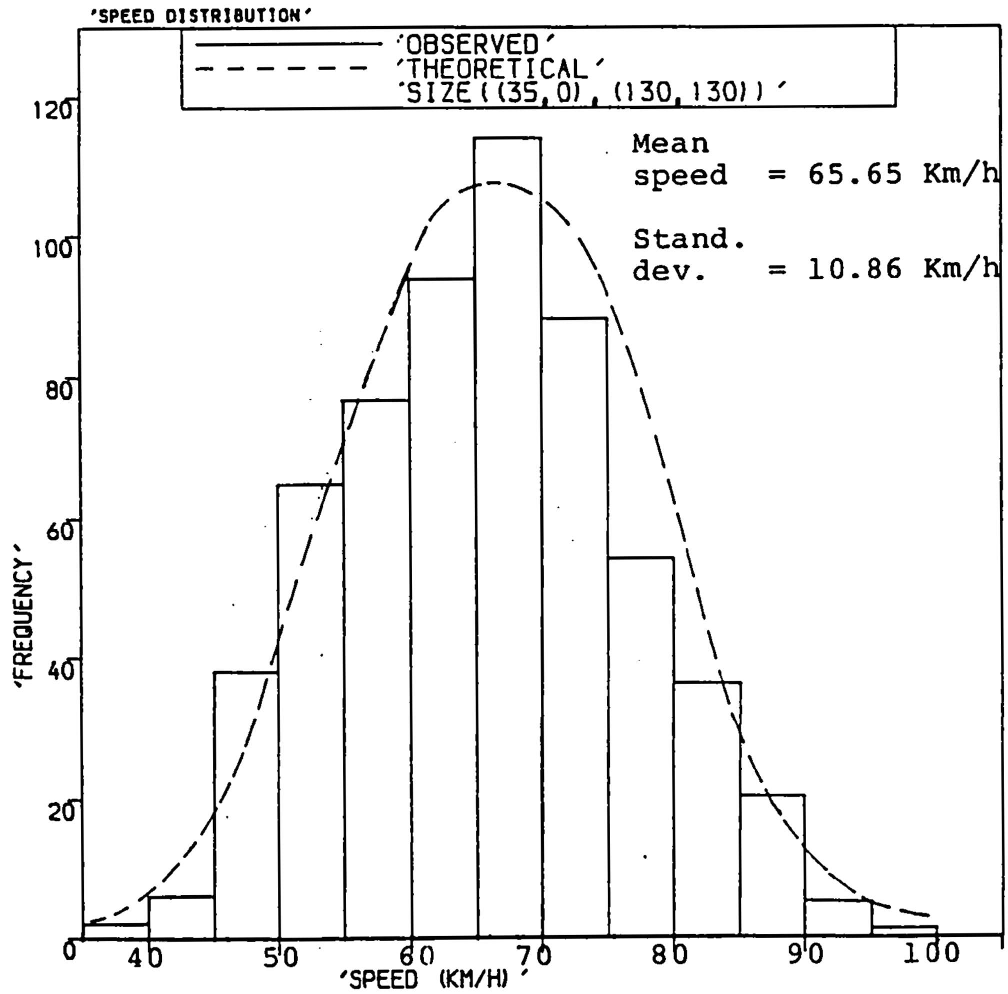


Figure 4.8.2-2 Speed Distribution (Heavy Goods Vehicles).

1	2	3	4	5	6	7
40 - 44.9	2	325	100	0.52	328.63	100.00
45 - 49.9	3	323	99.38	1.85	328.11	99.84
50 - 54.9	5	320	98.46	9.63	326.26	99.28
55 - 59.9	16	315	96.92	12.35	316.63	96.35
60 - 64.9	22	299	92.0	24.02	304.28	92.59
65 - 69.9	32	277	85.23	39.70	280.26	85.28
70 - 74.9	41	245	75.38	51.70	240.56	73.20
75 - 79.9	52	204	62.77	55.35	188.86	57.47
80 - 84.9	70	152	46.77	51.45	133.51	40.63
85 - 89.9	48	82	25.23	38.64	82.06	24.97
90 - 94.9	20	34	10.46	24.02	43.42	13.21
95 - 99.9	10	14	4.31	12.35	19.40	5.90
100 - 104.9	2	4	1.23	5.20	7.05	2.15
105 - 109.9	2	2	0.62	1.85	1.85	0.56

Table 4.8.2-3 Light goods vehicles speed distribution (near-side lane/off-peak periods).

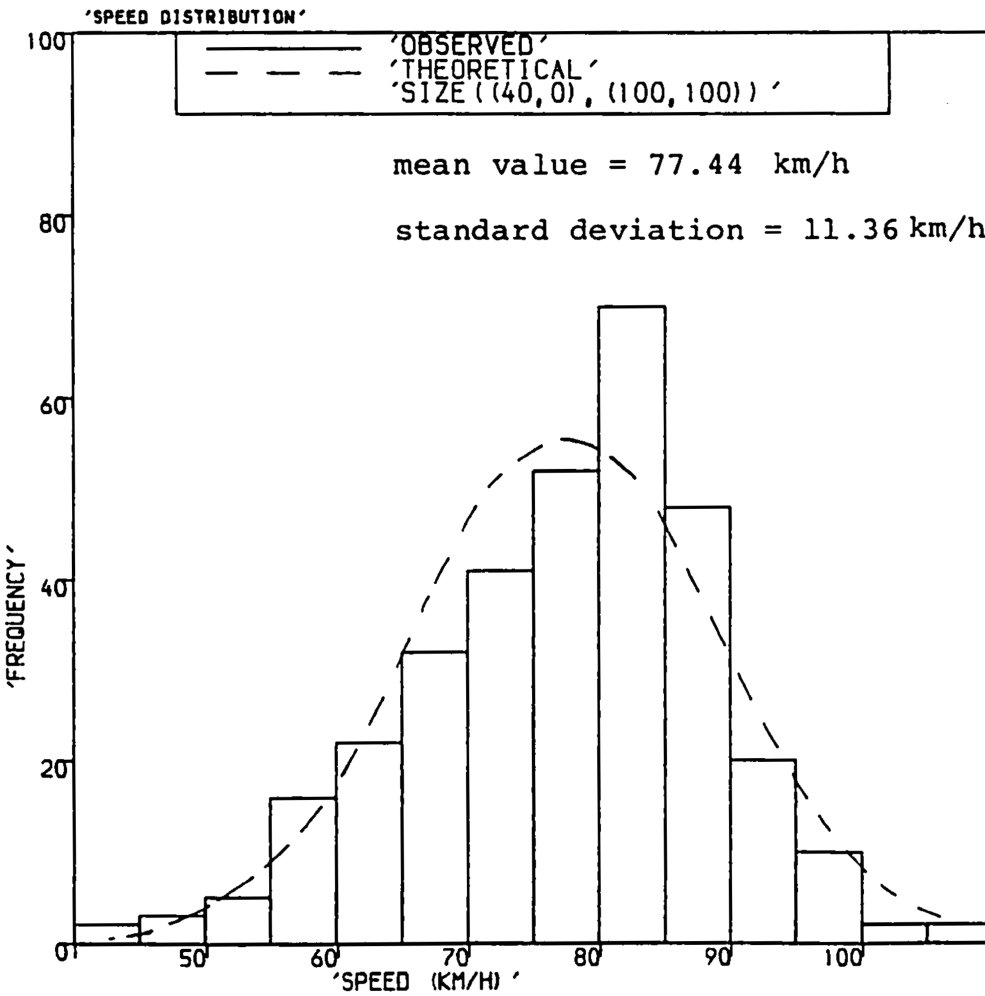


Figure 4.8.2-3 Speed distribution (light goods vehicles).

1	2	3	4	5	4	6
35 - 39.9	6	495	100	4.40	489.12	100.00
40 - 44.9	11	489	98.78	9.55	489.72	99.10
45 - 49.9	21	478	96.56	18.41	475.17	97.15
50 - 54.9	39	457	92.32	31.04	456.77	93.39
55 - 59.9	45	418	84.94	46.08	425.73	87.04
60 - 64.9	49	373	75.35	60.24	379.65	77.62
65 - 69.9	60	324	65.45	69.15	319.41	65.30
70 - 74.9	73	264	53.33	68.06	250.26	51.16
75 - 79.9	60	191	38.58	64.05	182.16	37.24
80 - 84.9	55	131	26.46	48.60	118.11	24.15
85 - 89.9	36	76	15.35	33.46	69.51	14.21
90 - 94.9	26	40	8.08	20.25	36.05	7.37
95 - 99.9	12	14	2.83	10.80	15.80	3.23
100 - 104.9	2	2	0.404	5.00	5.00	1.02

Table 4.8.2-4 Heavy goods vehicles speed distribution (near-side lane/off-peak periods).

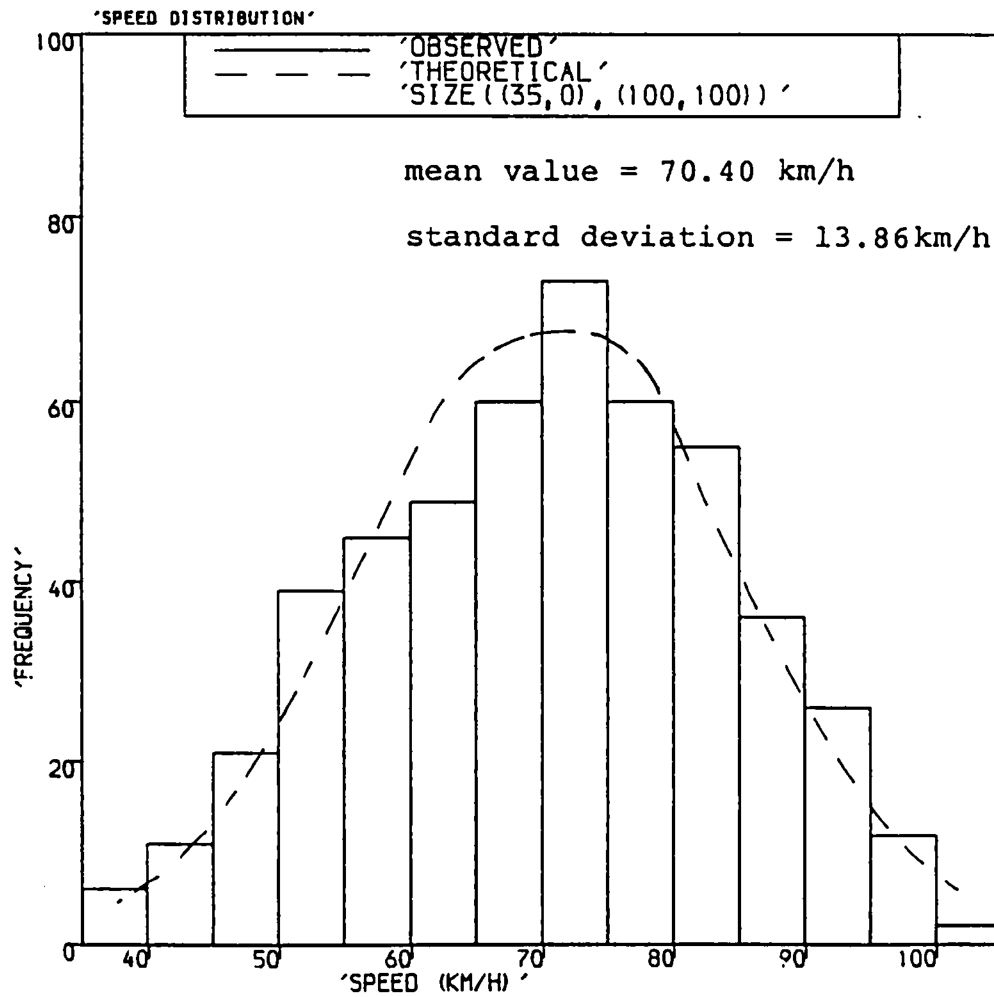


Figure 4.8.2-4 Spped distribution (heavy goods vehicles).

1	2	3	4	5	6	7
55 - 59.9	2	542	100	2.28	539.22	100.00
60 - 64.9	10	540	99.63	6.23	536.94	99.58
65 - 69.9	17	530	97.79	14.20	530.71	98.42
70 - 74.9	30	513	94.65	27.91	516.51	95.79
75 - 79.9	42	483	89.11	46.77	488.6	90.61
80 - 84.9	58	441	81.36	68.89	441.83	81.94
85 - 89.9	75	383	70.66	82.22	372.94	69.16
90 - 94.9	96	308	56.83	83.41	290.72	53.91
95 - 99.9	81	212	39.11	76.31	207.31	38.45
100 - 104.9	60	131	24.20	59.29	131	24.29
105 - 109.9	37	71	13.10	37.34	71.71	13.30
110 - 114.9	21	34	6.27	20.65	34.37	6.37
115 - 119.9	11	13	2.40	9.76	13.72	2.54
120 - 124.9	2	2	0.37	3.96	3.96	0.73

Table 4.8.2-5 Passenger cars speed distribution (far-side lane/peak periods).

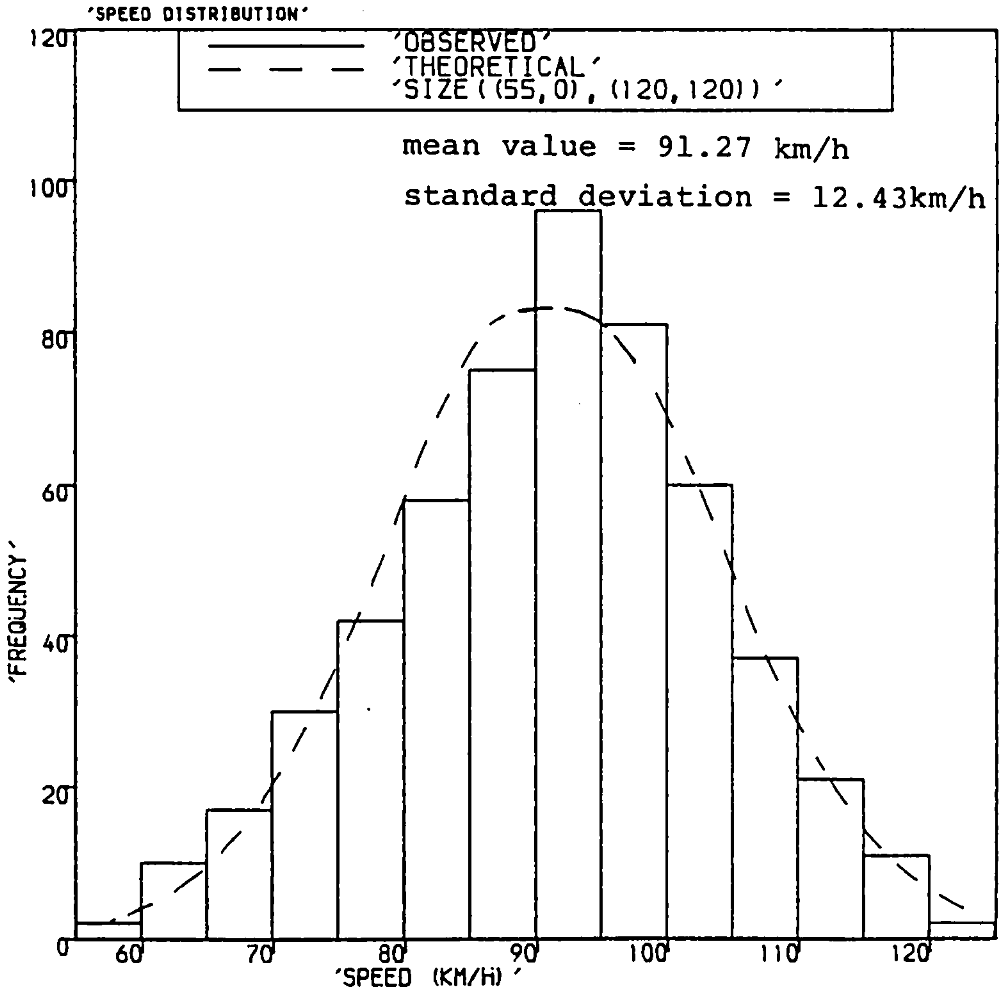


Figure 4.8.2-5 Speed distribution (passenger cars).

1	2	3	4	5	6	7
40 - 44.9	4	488	100	5.42	474.58	100.00
45 - 49.9	20	484	99.20	13.18	469.16	98.86
50 - 54.9	30	464	95.08	26.79	455.98	96.08
55 - 59.9	46	434	88.93	45.73	429.19	90.44
60 - 64.9	62	388	79.51	63.73	383.46	80.80
65 - 69.9	70	316	66.80	78.71	319.73	67.37
70 - 74.9	76	256	52.46	78.18	421.02	50.79
75 - 79.9	70	180	36.88	59.00	162.84	34.31
80 - 84.9	53	110	22.54	49.63	103.84	21.88
85 - 89.9	36	57	11.68	29.52	54.21	11.42
90 - 94.9	14	21	4.3	15.57	24.62	5.20
95 - 99.9	5	7	1.43	6.73	9.12	1.92
100 - 104.9	2	2	0.410	2.39	2.39	0.50

Table 4.8.2-6 Heavy and light goods vehicles speed distribution (far-side lane/peak periods).

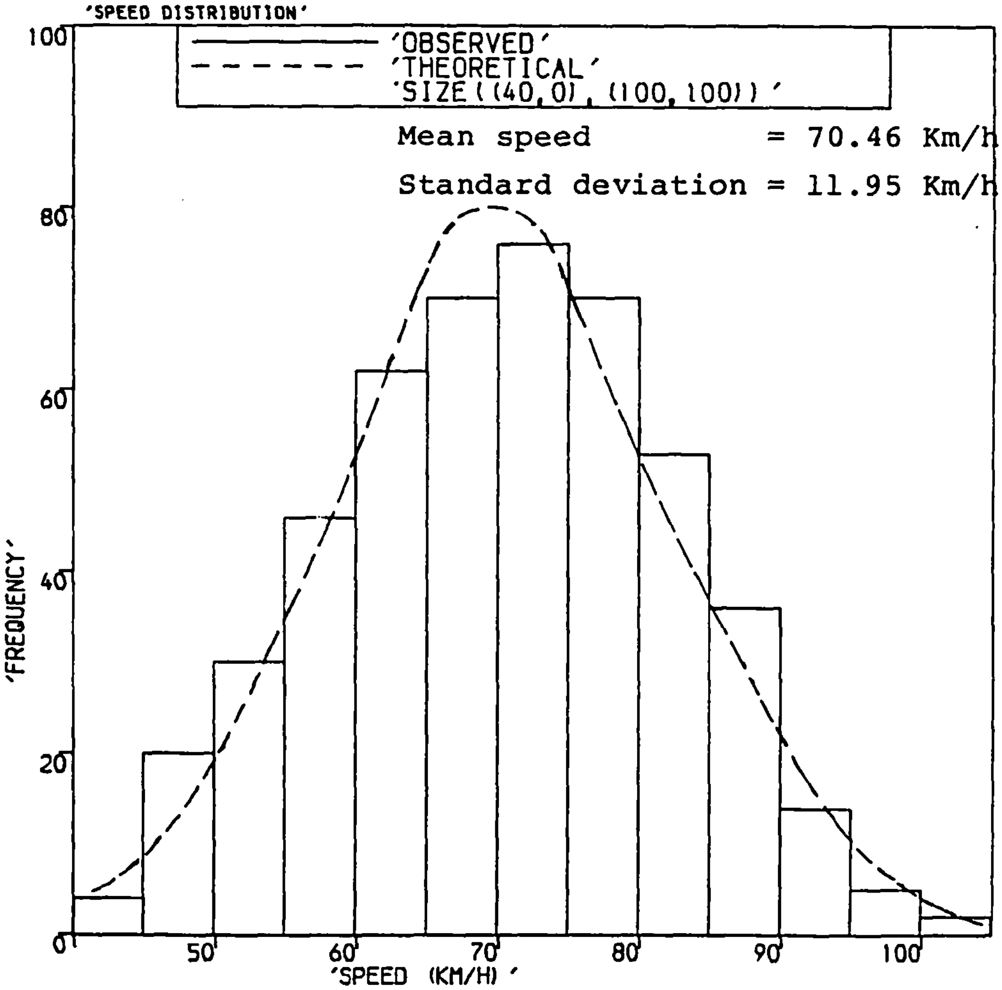


Figure 4.8.2-6 Speed Distribution (Heavy and Light Goods Vehicles).

1	2	3	4	5	6	7
55 - 59.9	4	412	100	2.80	437.18	100.00
60 - 64.9	12	408	99.03	7.00	434.38	99.36
65 - 69.9	15	396	96.12	13.68	427.38	97.76
70 - 74.9	23	381	92.48	24.88	413.7	94.63
75 - 79.9	31	358	86.89	38.64	388.82	88.94
80 - 84.9	42	327	79.37	51.95	350.18	80.10
85 - 89.9	66	285	69,17	60.60	298.23	68.22
90 - 94.9	71	219	53.15	59.78	237.63	54.36
95 - 99.9	56	148	35.92	54.01	177.85	40.68
100 - 104.9	39	92	22.33	41.24	123.84	28.33
105 - 109.9	22	53	12.86	27.32	82.60	18.89
110 - 114.9	18	31	7.5	15.33	55.28	12.64
115 - 119.9	10	13	3.15	7.95	23.28	5.33
120 - 124.9	3	3	0.73	3.46	7.95	1.82

Table 4.8.2-7 Passenger cars speed distribution (far-side lane/off-peak periods).

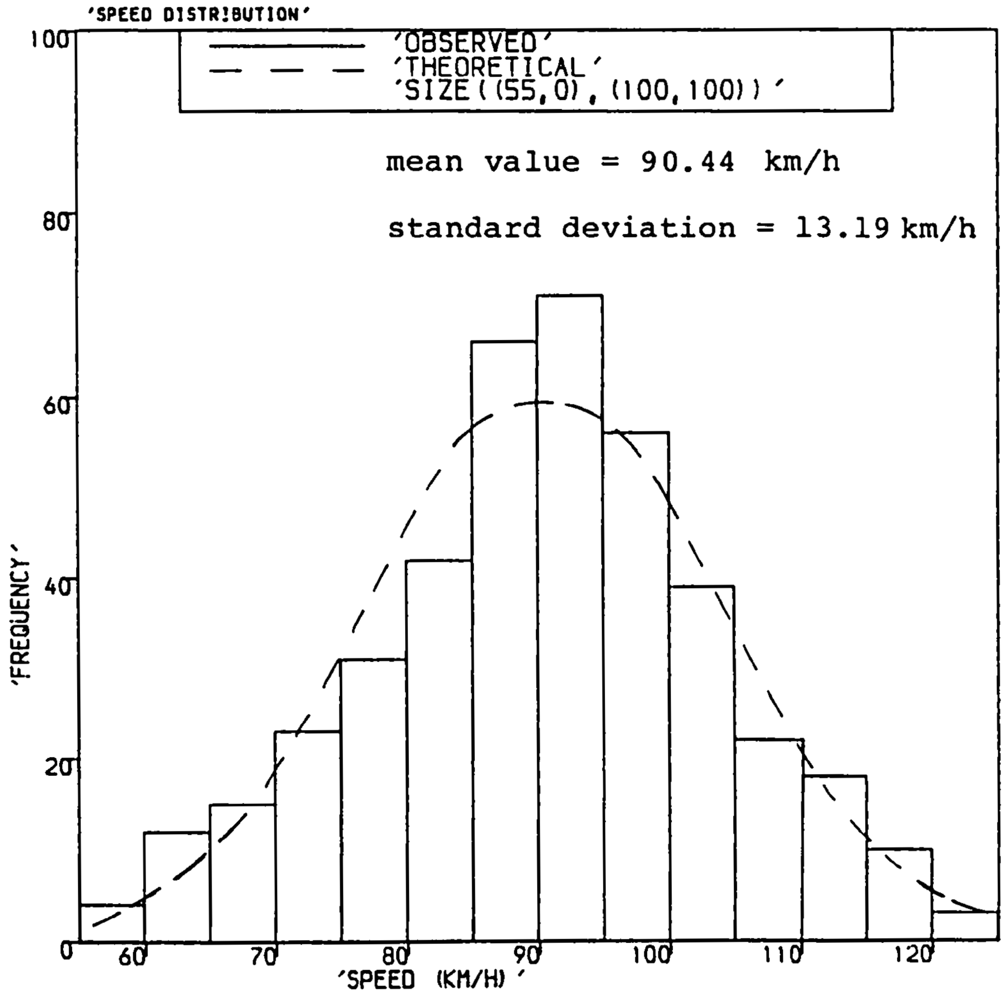


Figure 4.8.2-7 Speed distribution (passenger cars).

1	2	3	4	5	6	7
40 - 44.9	3	316	100	3.54	314.65	100.00
45 - 49.9	13	313	99.05	8.53	311.11	98.87
50 - 54.9	19	300	94.94	17.35	302.58	96.65
55 - 59.9	30	281	88.92	29.61	285.23	90.65
60 - 64.9	40	251	79.43	42.41	255.62	81.24
65 - 69.9	45	211	66.77	51.07	213.21	67.76
70 - 74.9	50	166	52.53	50.56	162.14	51.53
75 - 79.9	46	116	36.71	44.27	111.58	35.46
80 - 84.9	34	70	22.15	31.82	67.31	21.39
85 - 89.9	23	36	11.39	20.13	35.49	11.28
90 - 94.9	9	13	4.11	9.73	15.36	4.88
95 - 99.9	3	4	1.27	4.08	5.63	1.79
100 - 104.9	1	1	0.316	1.55	1.55	0.49

Table 4.8.2-8 Heavy and light goods vehicles speed distribution (far-side lane/off-peak periods).

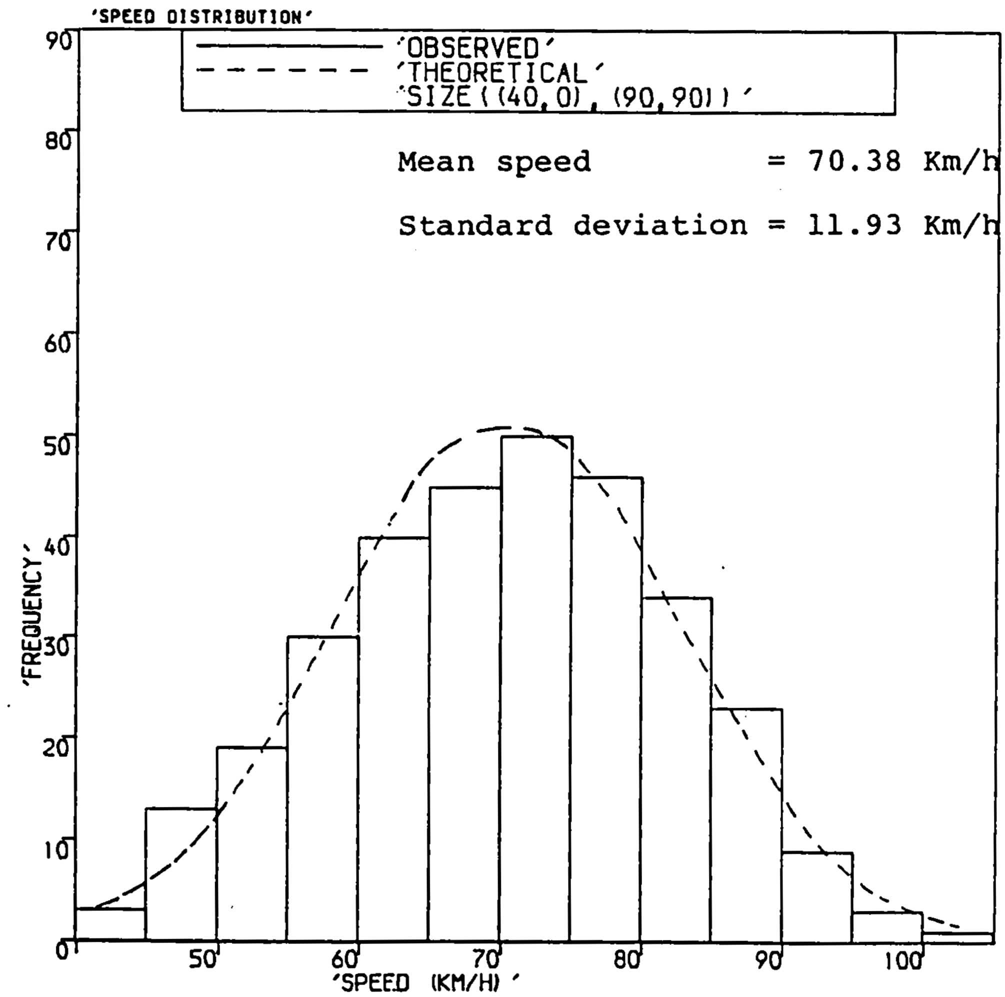


Figure 4.8.2-8 Speed Distribution (Heavy and Light Goods Vehicles).

	Near Side Lane				Far Side Lane			
	Peak Period		Off-Peak Period		Peak Period		Off-Peak Period	
Type of Vehicle	Light Goods Vehicle	Heavy Goods Vehicle	Light Goods Vehicle	Heavy Goods Vehicle	Cars	Light Goods Vehicle	Cars	Light Goods Vehicle
Total No. of Observation	464	600	325	495	542	488	412	316
Mean Speed (KM/H)	80.50	65.65	77.44	70.40	91.27	70.46	90.44	70.38
Standard Deviation (KM/H)	11.91	10.88	11.36	13.86	12.43	11.98	13.19	11.93
χ^2	12.35	13.332	24.369	26.155	9.200	9.730	22.650	4.610
$\chi^2_{0.95}$	22.362	21.026	22.362	22.362	22.362	21.026	22.362	21.026

Table 4.8.2-9 Summary of observed speeds for 2-lane motorway.

1	2	3	4	5	6	7
40 - 44.9	1	407	100.00	0.45	406.31	100.00
45 - 49.9	4	406	99.75	2.24	405.86	99.89
50 - 54.9	10	402	98.77	8.38	403.62	99.34
55 - 59.9	20	392	96.31	22.95	395.24	97.28
60 - 64.9	36	372	91.40	48.64	372.29	91.63
65 - 69.9	78	336	82.56	74.32	323.65	79.66
70 - 74.9	98	258	63.40	86.61	249.33	61.36
75 - 79.9	80	160	39.31	75.91	162.72	40.05
80 - 84.9	46	80	19.66	50.75	86.81	21.37
85 - 89.9	22	34	8.35	24.42	36.06	8.87
90 - 94.9	10	12	1.95	9.20	11.64	2.86
95 - 99.9	2	2	0.49	2.44	2.44	0.60

Table 4.8.2-10 Light goods vehicles speed distribution (near-side lane/peak periods).

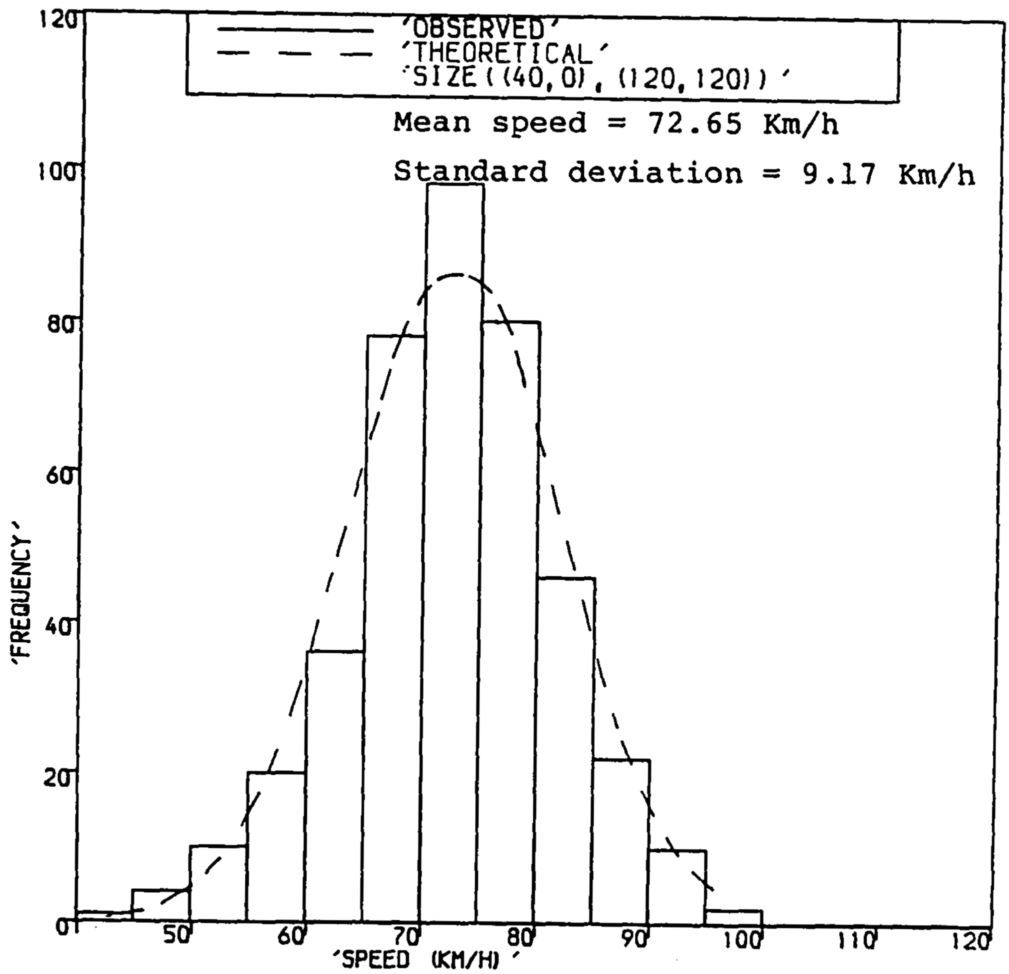


Figure 4.8.2-10 Speed Distribution (Light Goods Vehicles).

1	3	4	5	6	7	8
30 - 34.9	2	555	100.00	0.44	553.79	100.00
35 - 39.9	3	553	99.64	1.72	553.35	99.92
40 - 44.9	4	550	91.10	5.83	551.63	99.61
45 - 49.9	8	546	93.38	15.60	545.8	98.56
50 - 54.9	32	538	96.94	33.91	530.2	95.74
55 - 59.9	66	506	91.17	60.00	496.29	89.62
60 - 64.9	94	440	79.28	86.03	436.29	78.78
65 - 69.9	102	346	62.34	98.18	350.26	63.25
70 - 74.9	92	244	43.96	95.40	252.08	45.52
75 - 79.9	68	152	27.39	73.70	156.68	28.29
80 - 84.9	44	84	15.14	46.29	82.98	14.98
85 - 89.9	26	40	7.21	23.59	36.69	6.63
90 - 94.9	10	14	2.52	9.77	13.10	2.37
95 - 99.9	4	4	0.71	3.33	3.33	.60

Table 4.8.2-11 Heavy goods vehicles speed distribution (near-side lane/peak periods).

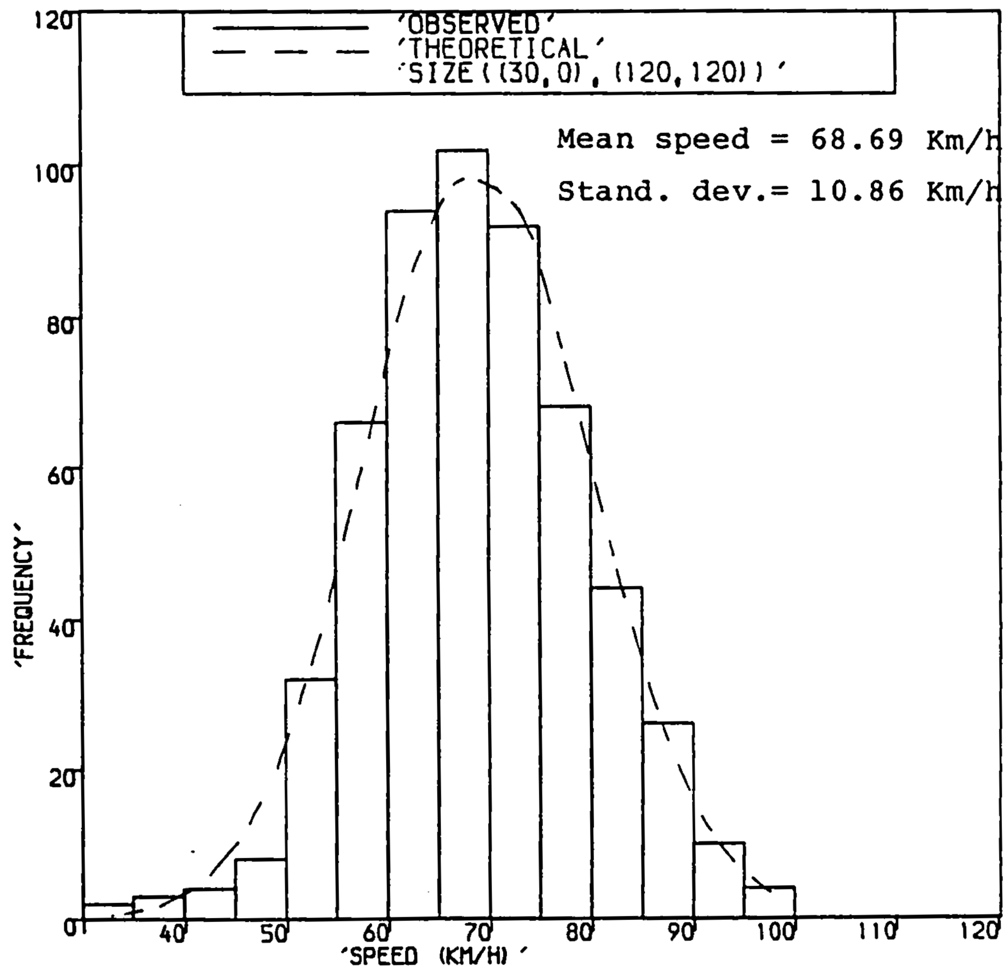


Figure4.8.2-11Speed Distribution (Heavy Goods Vehicles).

1	2	3	4	5	6	7
30 - 34.9	4	265	100.00	3.31	263.15	100.00
35 - 39.9	6	261	98.49	8.64	259.84	98.74
40 - 44.9	12	255	96.23	18.15	251.2	95.46
45 - 49.9	22	243	91.70	30.95	233.05	88.56
50 - 54.9	52	221	83.40	41.82	202.1	76.80
55 - 59.9	68	169	63.77	47.12	160.28	60.91
60 - 64.9	48	101	38.11	44.39	113.16	43.00
65 - 69.9	28	53	20.00	27.80	68.77	26.13
70 - 74.9	14	25	9.43	15.65	40.97	15.57
75 - 79.9	6	11	4.15	9.99	15.32	5.82
80 - 84.9	3	5	1.89	3.95	5.33	2.03
85 - 89.9	2	2	0.755	1.38	1.38	0.52

Table 4.8.2-12 Light goods vehicles speed distribution (near-side lane/off-peak periods).

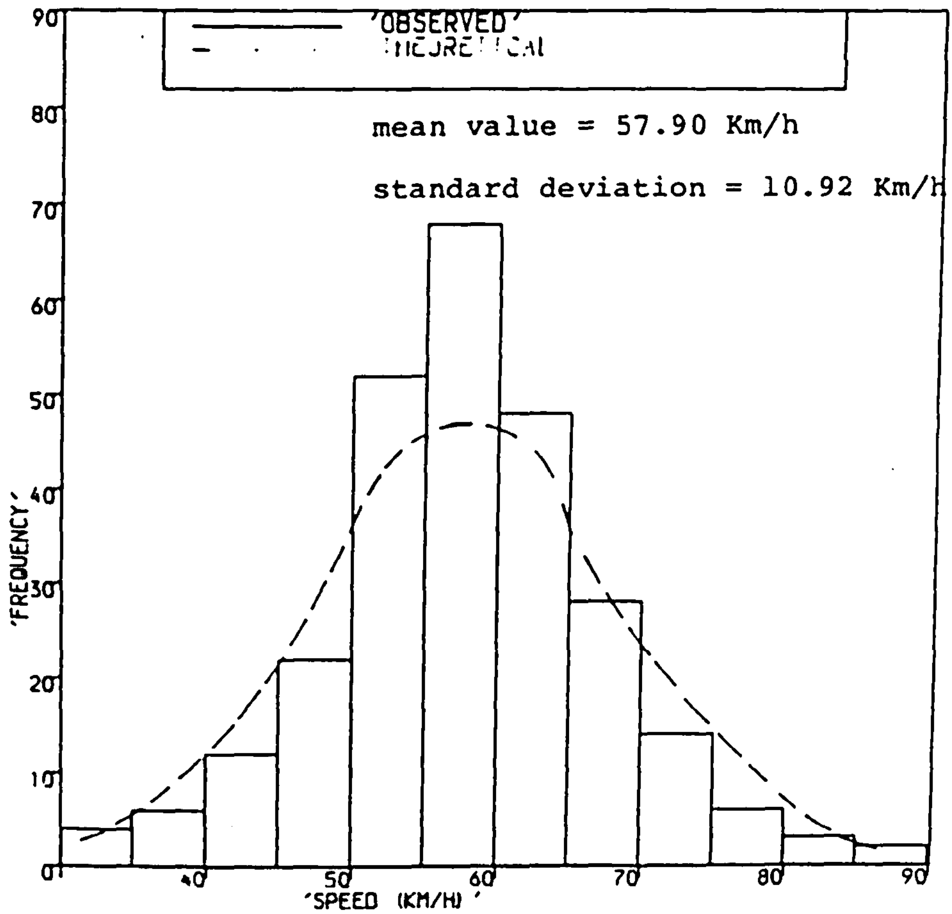


Figure 4.8.2-12 Speed distribution (light goods vehicles).

1	2	3	4	5	6	7
30 - 34.9	4	393	100.00	2.004	396.27	100.00
35 - 39.9	8	384	98.98	6.33	394.27	99.50
40 - 44.9	14	381	96.95	15.84	387.94	97.90
45 - 49.9	25	367	93.38	32.07	372.10	93.90
50 - 54.9	48	342	87.02	57.353	340.03	85.81
55 - 59.9	76	293	74.62	68.85	282.49	71.29
60 - 64.9	82	218	55.47	72.47	213.64	53.91
65 - 69.9	72	136	34.60	61.347	141.17	35.62
70 - 74.9	32	64	16.28	42.52	79.82	20.14
75 - 79.9	16	32	8.14	22.636	37.30	9.41
80 - 84.9	9	16	4.07	10.06	14.66	3.70
85 - 89.9	4	7	1.78	3.42	4.60	1.16
90 - 94.9	2	3	0.76	0.98	1.18	.29
95 - 99.9	1	1	0.25	0.196	0.196	.05

Table 4.8.2-12 Heavy goods vehicles speed distribution (near-side lane/off peak periods).

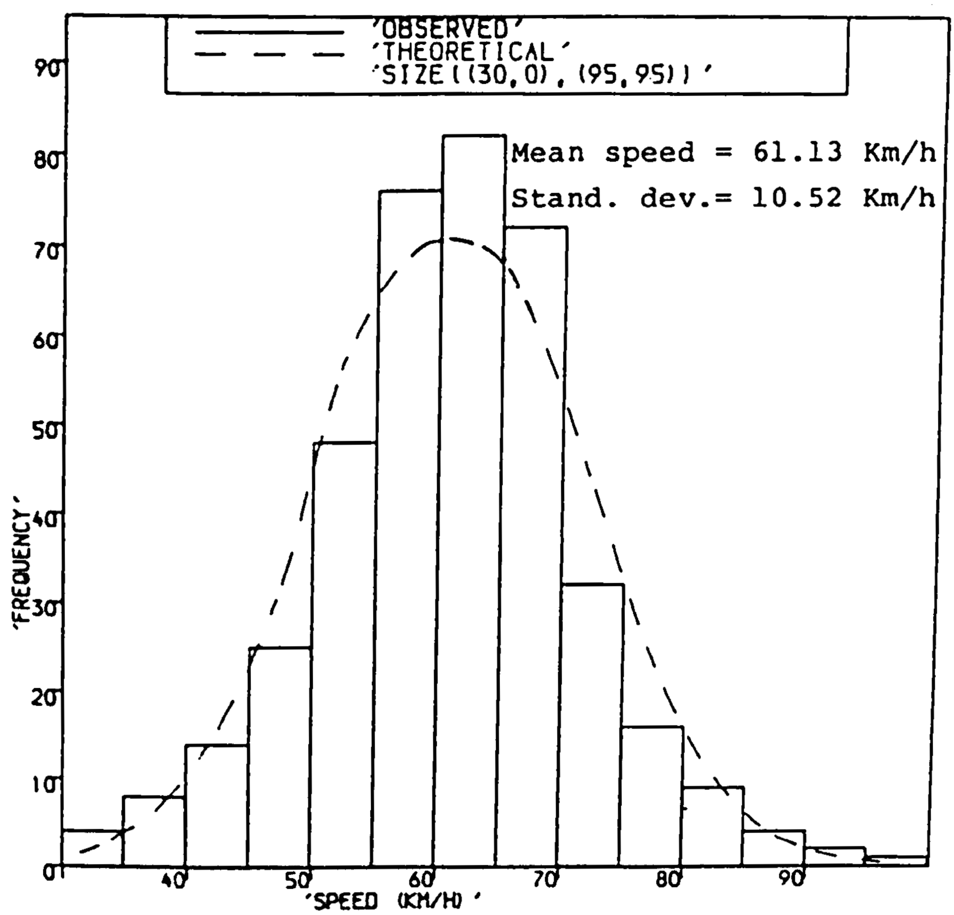


Figure 4.8.2-13 Speed Distribution (Heavy Goods Vehicles).

1	2	3	4	5	6	7
35 - 39.9	2	242	100.00	0.51	237.88	100.00
40 - 44.9	3	240	99.17	1.84	237.37	99.79
45 - 49.9	6	237	97.93	5.49	235.53	99.01
50 - 54.9	12	231	95.07	13.07	230.04	96.70
55 - 59.9	20	219	90.50	24.80	216.97	91.21
60 - 64.9	30	199	82.22	35.14	192.17	80.78
65 - 69.9	48	169	69.83	45.33	157.03	66.01
70 - 74.9	57	121	50.00	43.66	111.70	46.96
75 - 79.9	32	64	26.45	33.52	68.04	28.60
80 - 84.9	18	32	13.22	20.55	34.52	14.51
85 - 89.9	10	14	5.79	10.07	13.97	5.87
90 - 94.9	4	4	1.65	3.90	3.90	1.64

Table 4.8.2-14 Light goods vehicles speed distribution (middle lane/peak periods).

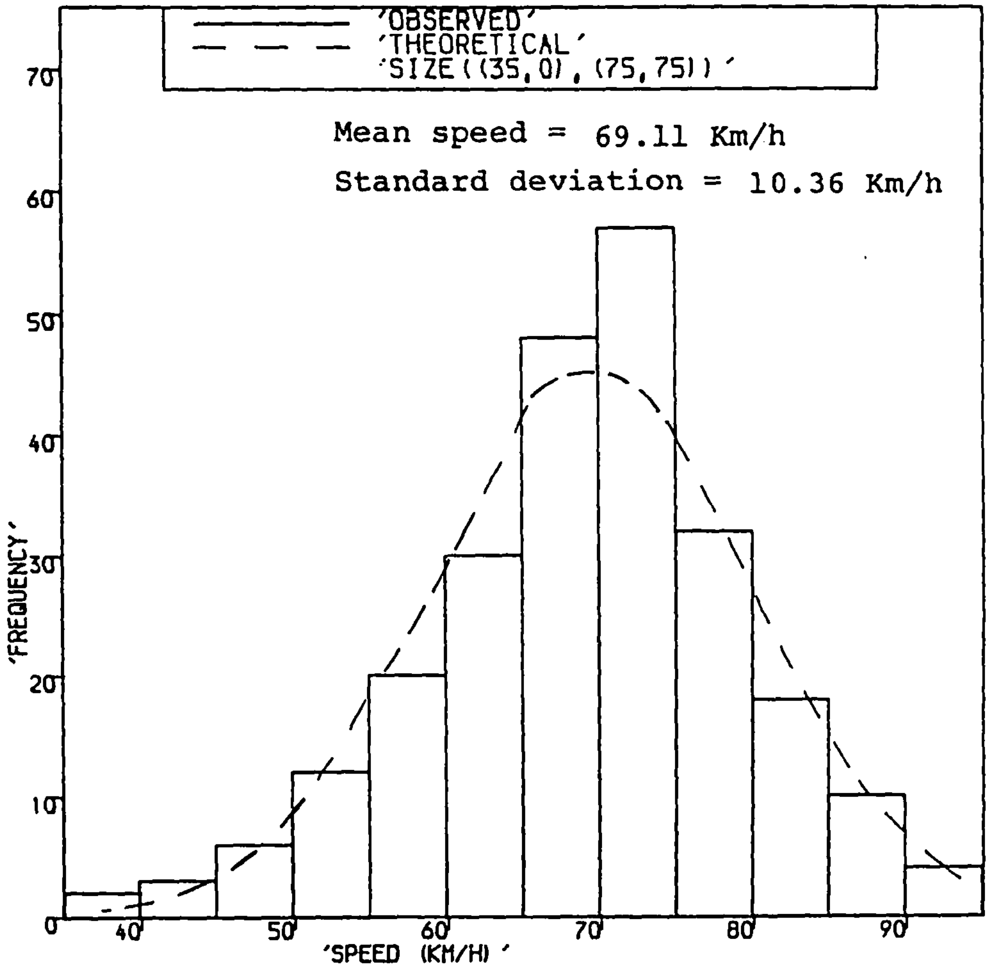


Figure4.8.2-14 Speed Distribution (Light Goods Vehicles).

1	2	3	4	5	6	7
50 - 54.9	3	489	100.00	1.32	490.21	100.00
55 - 59.9	4	486	99.39	3.81	488.89	99.73
60 - 64.9	10	482	98.57	9.83	485.08	98.95
65 - 69.9	20	472	96.52	21.27	475.25	96.95
70 - 74.9	34	452	92.43	43.52	453.98	92.61
75 - 79.9	58	418	85.48	58.80	410.46	83.73
80 - 84.9	82	360	73.62	75.45	351.59	71.72
85 - 89.9	96	278	56.85	79.41	276.14	56.33
90 - 94.9	78	182	37.22	73.94	196.73	40.13
95 - 99.9	42	104	21.17	56.48	122.79	25/05
100 - 104.9	30	74	15.13	36.28	66.31	13.53
105 - 109.9	20	40	8.18	19.56	30.03	6.13
110 - 114.9	12	20	4.09	5.97	10.47	2.14
115 - 119.9	6	8	1.64	3.33	4.50	0.92
120 - 124.9	2	2	0.41	1.17	1.17	0.24

Table 4.8.2-15 Passenger cars speed distribution (middle lane/peak periods).

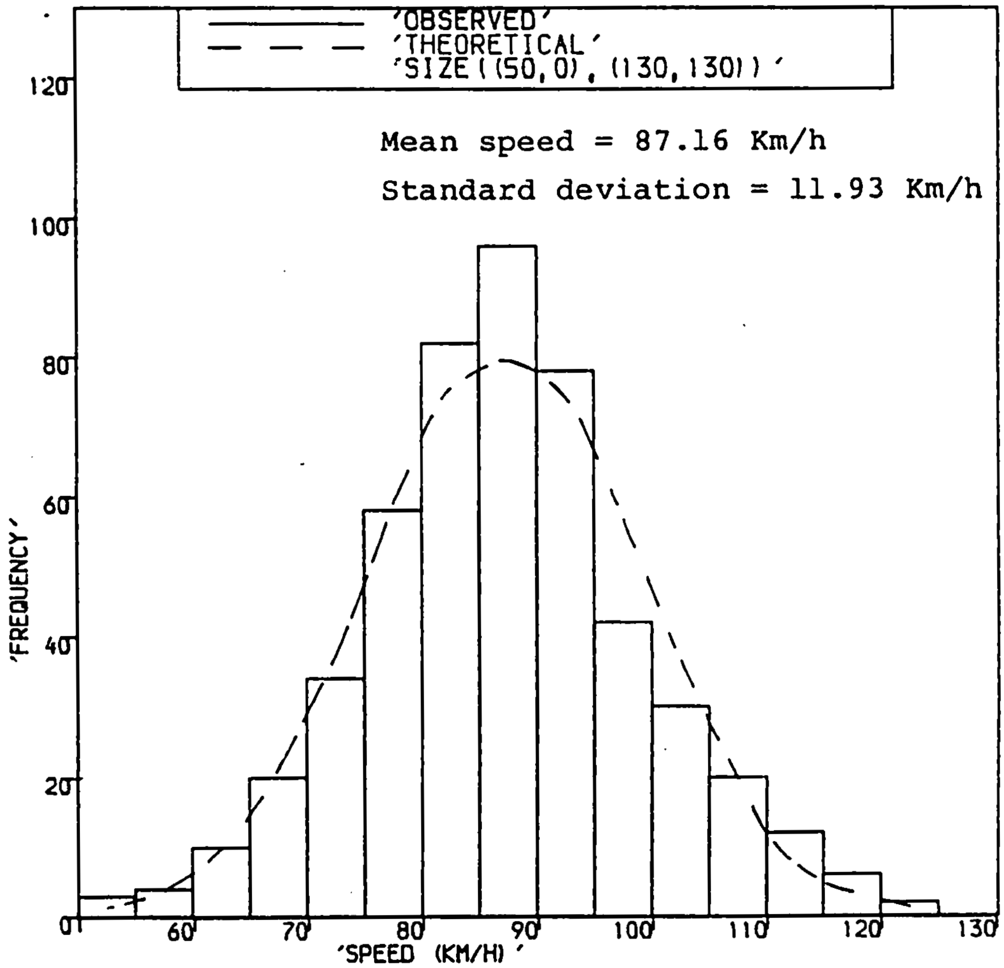


Figure4.8.2-15Speed Distribution (Passenger Cars).

1	2	3	4	5	6	7
35 - 39.9	2	177	100.00	0.34	178.15	100.00
40 - 44.9	3	175	98.87	1.19	177.81	99.81
45 - 49.9	4	172	97.18	3.79	176.62	99.14
50 - 54.9	6	168	94.92	9.51	172.83	97.01
55 - 59.9	14	162	91.53	18.69	163.32	91.68
60 - 64.9	26	148	83.62	28.78	144.63	81.18
65 - 69.9	35	122	68.93	33.95	115.85	65.03
70 - 74.9	38	87	49.15	32.80	81.90	45.97
75 - 79.9	32	49	27.68	24.32	49.10	27.56
80 - 84.9	12	17	9.60	14.12	24.78	13.91
85 - 89.9	3	5	2.82	8.20	10.66	5.98
90 - 94.9	2	2	1.13	2.46	2.46	1.38

Table 4.8.2-16 Light goods vehicles speed distribution (middle lane/off-peak periods).

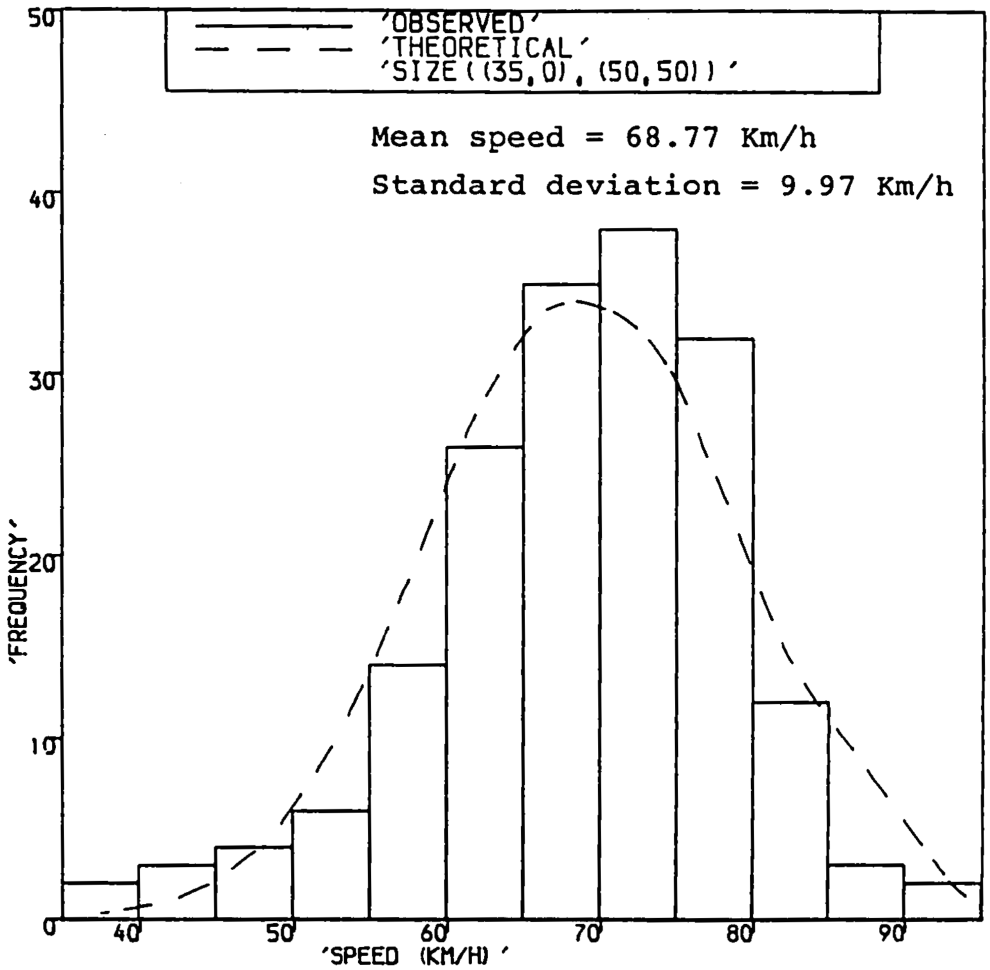


Figure 4.8.2-16 Speed Distribution (Light Goods Vehicles).

1	2	3	4	5	6	7
50 - 54.9	2	405	100.00	1.13	404.29	100.00
55 - 59.9	4	403	99.51	3.32	403.16	99.72
60 - 64.9	8	399	98.52	8.1	399.84	98.90
65 - 69.9	15	391	96.54	16.73	391.74	96.90
70 - 74.9	24	376	92.84	28.80	375.01	92.75
75 - 79.9	37	352	86.91	74.55	346.21	85.63
80 - 84.9	72	315	77.78	57.83	346.21	74.61
85 - 89.9	75	243	60.00	62.49	243.83	60.31
90 - 94.9	59	168	41.48	60.83	181.34	44.85
95 - 99.9	42	109	26.91	49.83	120.51	29.81
100 - 104.9	29	67	16.54	34.23	70.68	17.48
105 - 109.9	18	38	9.38	20.21	36.45	9.02
110 - 114.9	10	20	4.94	10.21	16.24	4.02
115 - 119.9	6	10	2.47	4.33	6.03	1.49
120 - 124.9	4	4	0.99	1.70	1.70	0.42

Table 4.8.2-17 Passenger cars speed distribution (middle lane/off-peak periods).

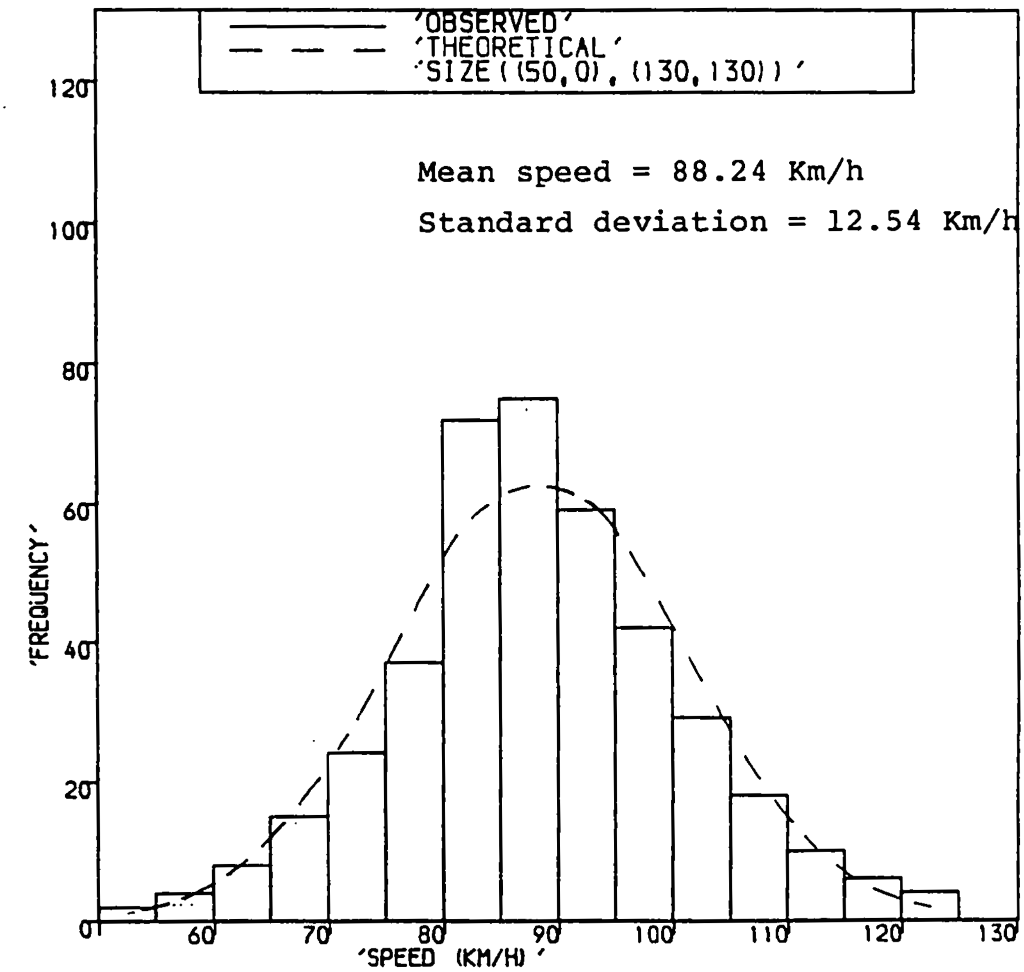


Figure4.8.2-17 Speed Distribution (Passenger Cars).

1	2	3	4	5	6	7
70 - 74.9	3	576	100.00	3.57	572.76	100.00
75 - 79.9	12	573	99.48	10.37	569.19	99.38
80 - 84.9	30	561	97.40	24.88	558.82	97.57
85 - 89.9	52	531	92.19	49.80	533.94	93.22
90 - 94.9	75	479	83.16	77.76	483.95	84.49
95 - 99.9	90	404	70.14	101.89	406.19	70.92
100 - 104.9	100	314	54.51	101.49	304.3	53.13
105 - 109.9	90	214	37.15	89.15	202.81	35.41
110 - 114.9	72	124	21.53	59.73	113.42	19.80
115 - 119.9	37	52	9.03	33.18	53.69	9.37
120 - 124.9	11	15	2.60	15.15	20.51	3.58
125 - 129.9	4	4	0.69	5.36	5.36	0.94

Table 4.8.2-18 Passenger cars speed distribution(far-side lane/ peak periods).

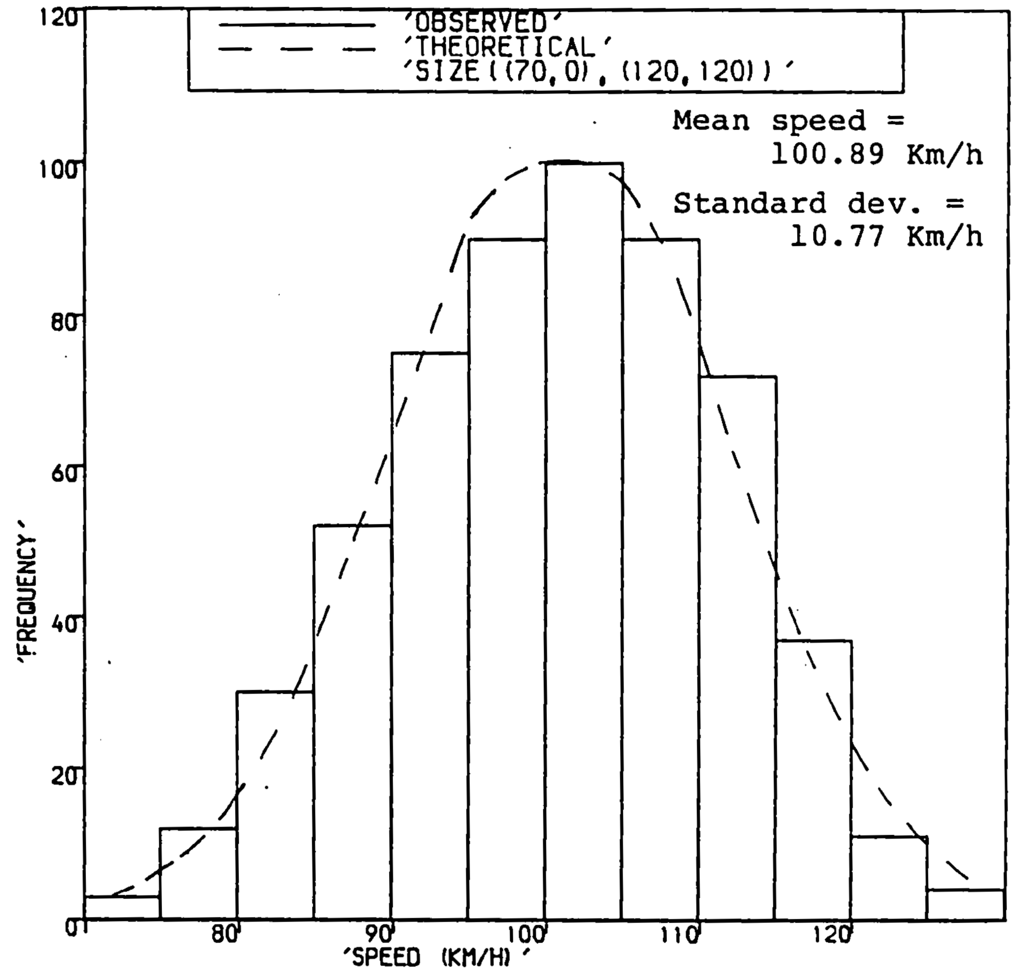


Figure4.8.2-18Speed Distribution (Passenger Cars).

1	2	3	4	5	6	7
70 - 74.9	2	415	100.00	0.789	413.50	100.00
75 - 79.9	4	413	99.52	3.28	412.71	99.81
80 - 84.9	12	409	98.55	10.333	409.43	99.02
85 - 89.9	24	397	95.66	26.311	399.10	96.52
90 - 94.9	42	373	89.88	50.67	372.79	90.15
95 - 99.9	78	331	79.76	74.99	322.12	77.90
100 - 104.9	87	253	60.96	83.54	247.13	59.76
105 - 109.9	74	166	40.00	73.37	163.59	39.56
110 - 114.9	54	92	22.17	50.26	90.22	21.82
115 - 119.9	28	38	9.16	26.31	39.96	9.66
120 - 124.9	8	10	2.41	10.22	13.65	3.30
125 - 129.9	2	2	0.48	3.32	3.32	.80

Table 4.8.2-19 Passenger cars speed distribution(far-side lane/off-peak periods).

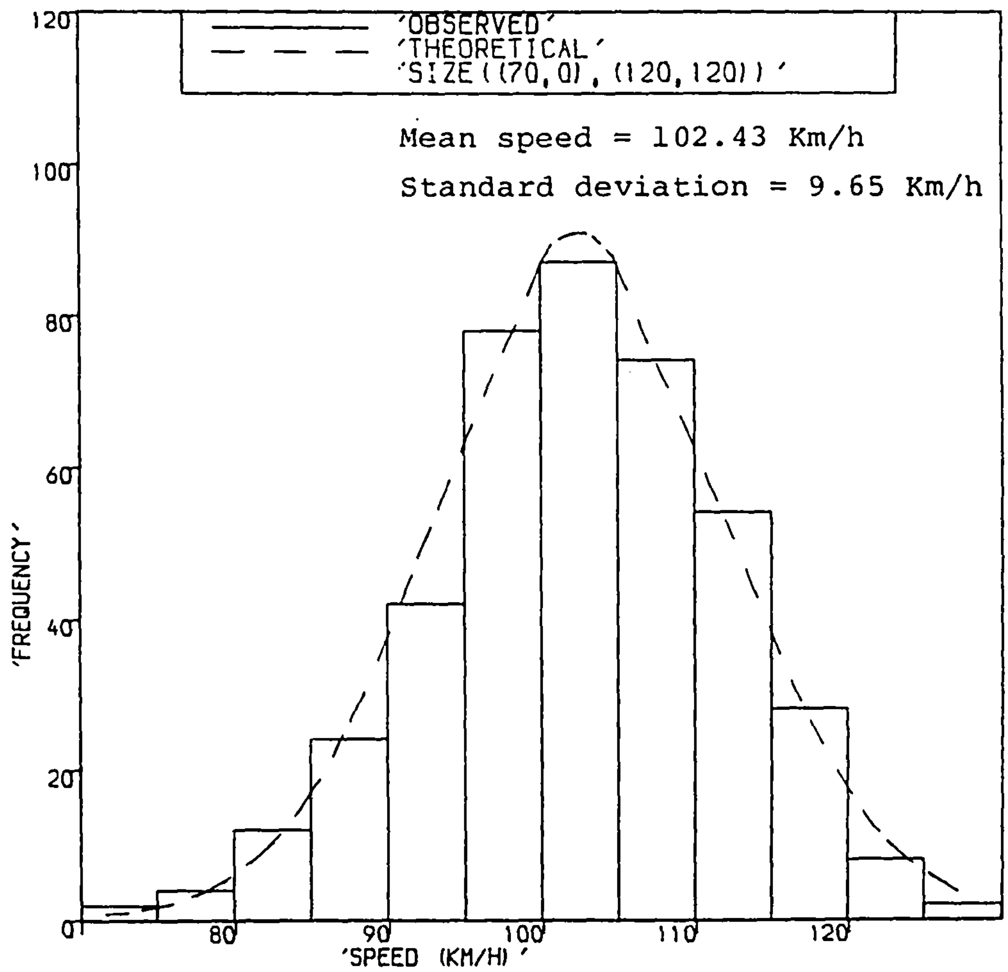


Figure 4.8.2-19 Speed Distribution (Passenger Cars).

	Near-side Lane				Middle Lane				Far-side Lane	
	Peak Period		Off-Peak Period		Peak Period		Off-Peak Period		Peak Period	Off-Peak Period
Type of Vehicle	Light Goods Vehicle	Heavy and Articul. Goods Vehicle	Light Goods Vehicle	Heavy and Articul. Goods Vehicle	Light Goods Vehicle	Cars	Light Goods Vehicle	Cars	Cars	Cars
Total No. of Observation	407	555	265	393	242	489	177	405	576	415
Mean Speed (KM/H)	72.65	68.69	57.90	61.13	69.11	87.16	68.77	88.24	100.89	102.43
Standard Deviation (KM/H)	9.17	10.86	10.92	10.52	10.38	11.93	10.00	12.54	10.77	9.65
χ^2	8.766	13.417	25.040	18.758	11.522	22.207	20.594	15.132	7.452	5.648
$\chi^2 0.95$	19.675	22.362	19.675	22.362	19.675	23.685	19.675	23.658	19.675	19.675

Table 4.8.2-20 Summary of observed speeds for 3-lane Motorway.

higher speeds than heavier vehicles.

ii - Speed mean values of passenger cars and light goods vehicles travelling in the middle lane have almost the same values for peak and off-peak periods.

iii - Vehicles travelling in the slow lane (light goods vehicles, heavy and articulated goods vehicles) have lower speed mean values in off-peak periods than in the peak periods. A possible explanation for this is the attitude of the drivers. The driver of a commercial vehicle is generally more concerned about time (i.e. speed) than the driver of a car during peak periods since time means money to the lotty driver. Therefore, he is more likely to be travelling as fast as possible.

iv - Speed mean values are inversely proportional to size of vehicles. This is apparent for both near side and middle lanes, and is due to vehicle type movement capability.

4.9 - Vehicle type distribution

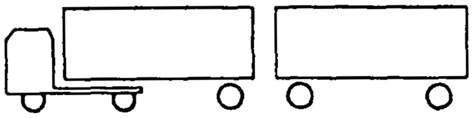
Percentages of heavy commercial vehicles of the total flow on rural highways are required for road design and capacity calculation, because of their impact on average speeds and on total traffic flow performance, especially as they relate to grade-climbing capabilities. Also their increasing sizes (length and width) dectate requirements for the dimensions of the roadway; lane width must accommodate the widest vehicle normally using the road; the longest vehicle must be able to negotiate the sharpest curve. The variation in percentages of heavy commercial vehicles during the hours of observation per lane of travel was noted. Heavy commercial vehicles were classified according to their significant numbers and due to their wide range of sizes and axles arrangement, the classification was according to number of axles. Figure (4. 9 -1) shows the different arrangements of heavy commercial vehicles observed with their average lengths in meters. Table (4. 9 -2) gives data of the observed number of heavy commercial vehicles with their corresponding number of axles. The observed number of heavy vehicles versus the number of axle arrangements were plotted as shown in figure (4. 9 -2), which shows that the highest number of axles per heavy commercial vehicles are the ones with 4-axles (two-axle tractor and two-axle trailer).

Traffic flow composition was investigated and vehicle type distributions in each lane were recorded and data are presented in table (4.9 -1). This data is also represented by figure (4. 9-1).

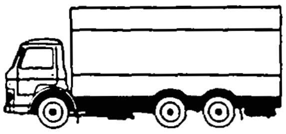
Vehicle Type		Two-lane Motorway		Three-lane Motorway		
		Near side lane(VPH)	Far side lane(VPH)	Near side lane(VPH)	Middle lane(VPH)	Far side lane(VPH)
Cars	No.	247	628	419	984	578
	(%)	(17)	(62)	(30)	(69)	(90)
Light and medium size goods vehicles	No.	339	210	255	219	43
	(%)	(23)	(20)	(19)	(16)	(7)
Heavy goods vehicles	No.	648	129	493	136	15
	(%)	(43)	(13)	(36)	(10)	(2)
Articulated goods vehicles	No.	217	46	172	56	-
	(%)	(15)	(45)	(13)	(4)	
Others	No.	26	4	32	21	6
	(%)	(2)	(0.5)	(2)	(1)	(1)
Total (VPH)		1477	1017	1371	1416	642

Table 4. 9 - 1 Vehicle type distribution per lane per hour (peak-periods).

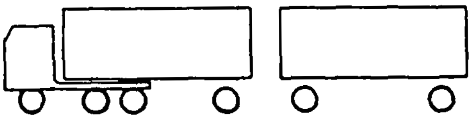
The observed distribution of heavy commercial vehicles classified by number of axles in this study agrees with the latest report published by the Department of Transport (DTp) (22). It is reported that the freight moved by heavy lorries in Great Britain increased by about 2.5 per cent between 1984



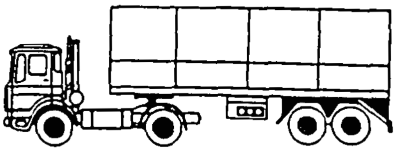
Trailer (Double)
5-axles (Av. length 14.45m)



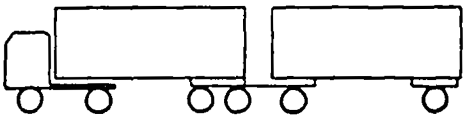
Single Unit HGV
3-axles (Av. length 7.75m)



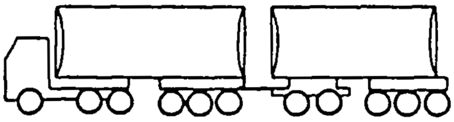
Trailer (Double)
6-axles (Av. length 15.0m)



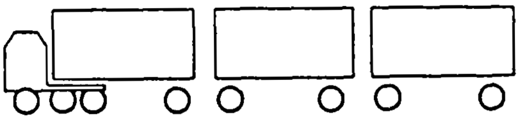
Articulated G.V.
4-axles (Av. length 10.67m)



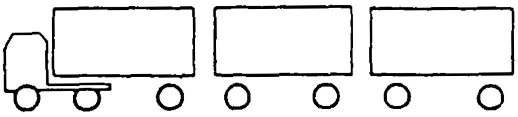
Draw Bar Trailer-Tandem axle
6-axles (Av. length 15.15m)



Double Tanker 11-axles
(Av. length 15.10m)



Triple Trailer Combination
8-axles (Av. length 15.40m)



Triple Trailer Combination
7-axles (Av. length 15.40m)

Figure 4. 9 -1 Heavy Commercial Vehicles Classes Observed.

Number of Axles	Observed number of heavy vehicles
2	658
3	715
4	824
5	550
6	307
7	243

Table 4.9 -2 Heavy commercial vehicles in number of axle arrangement.

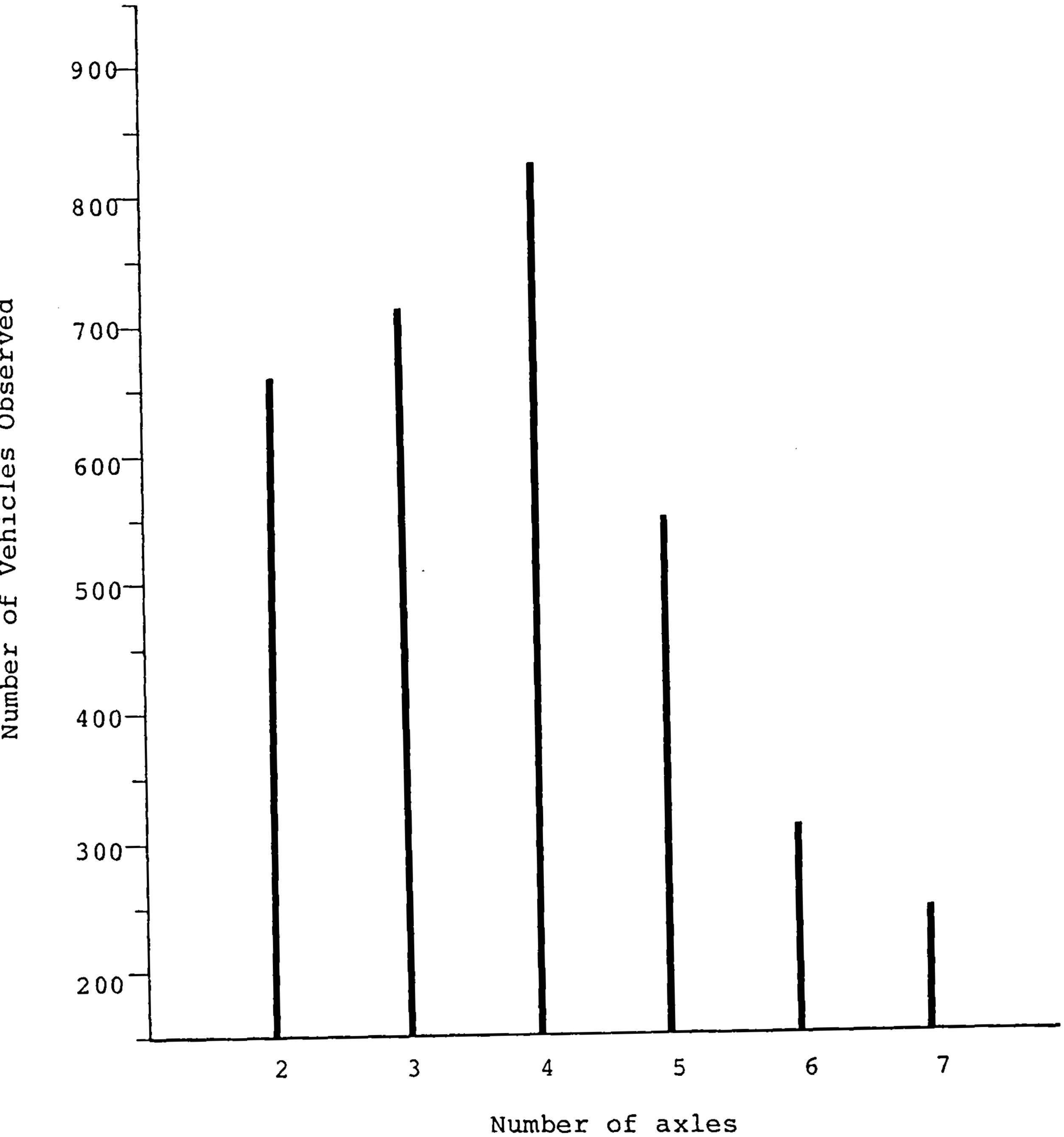


Figure 4. 9 -2 Heavy Commercial Vehicle Distribution According to their Total Number of Axles.

and 1985. Most of this was carried by 38 tonne vehicles and there were signs of a shift towards the use of haulage transport fleets. The rise in road freight activity during 1985 showed a significant increase where 1.4 billion tonnes were carried, an average of over 3,000 tonnes per vehicle. The number of four and five-axle, 3 tonne articulated vehicles showed the highest increase on the roads. For example the number of five-axle articulated goods vehicles rose from 19,000 at the end of 1984 to 26,000 at the end of 1985. Although this is only about 6 per cent of the stock of HGVs, 38 tonne vehicles accounted for over a quarter of the total of tonnes moved, and they travelled, on average, over 80,000 km (50,000 miles) in the year compared with an average of 37,000 km (23,000 miles) for all HGV's.

The above figures indicate the trend towards higher activity of heavy commercial vehicles on the roads and a higher number of trips undertaken. This increase will affect traffic flow performance and roadway capacity which will be discussed in section 4.11 of this chapter.

4.10 - Passenger car unit equivalencies for two-lane rural highways

4.10-1 - Vehicle type overtaking criteria

Overtaking procedure covers the movements of each vehicle type from the entry zone of analysis described as "point 10" up to its exit point "1" covering a total distance of 270m as shown in figure (4.7.1-1). The section of the road considered for analysis is assumed to represent an ideal situation for studying the overtaking procedure. It is the intention of this study to include the typical behaviour characteristics of the vast majority of vehicle and their operators.

Towards this end, certain general rules were established for the analysis procedure:

1. Vehicles enter and exit the zone of analysis at their own desired speeds,
2. Vehicles enter the zone of analysis in accordance with prescribed random distribution,
3. There is no restraint or forced flow condition,
4. There is no obstruction from stationary vehicles,
5. Vehicles are free to change lane and overtake within the zone of analysis.

Overtaking manoeuvre decisions by drivers are considered for passenger cars only. Passenger cars are the base vehicle by which to measure other vehicle types. Four vehicle type categories were considered for overtaking manoeuvre analysis:

1. Passenger cars
2. Light goods vehicles (LGV)
3. Heavy goods vehicles (HGV)
4. Articulated goods vehicles (AGV)

Passenger car (overtaking vehicles) speeds and the number of overtakings performed within the analysis zone were recorded. Also the overtaken vehicles' speeds were noted.

4.10.2 - Vehicle types equivalents

Data obtained from film analysis of the overtaking procedure between faster moving vehicles (passenger cars) and slower moving vehicles (light, heavy and articulated goods vehicles) were used to calculate their passenger car unit equivalencies.

To simplify the computation of passenger car unit equivalencies using equations (4.5.3-4) and (4.5.3-5) developed in section (4.5.3) of this chapter, special tables were introduced (e.g. table 4.10.2-1). These tables were designed to show the calculation of the parameters in the above equations and they show the following:-

i) the first column represents the number of slower moving vehicles (overtaken vehicles) in each average speed class.

ii) the second column represents average speed classes of slower moving vehicles and their inverses.

iii) the top line shows the number of faster moving vehicles (overtaking vehicles) in each average speed class.

iv) the second line shows the average speed classes of faster moving vehicles and their inverses.

v) in line three column three, to line thirteen column thirteen, the top figures shown in each line represent the inverse of S_1 multiplied by S_2 and the bottom figures in each line represent the "N" values which are obtained using the following:

$$N = Q_1 \; Q_2 \quad \left(\frac{S_2 - S_1}{S_1 \times S_2} \right)$$

The result of each line is given in the last column of the table and the summation of these values will give the total number of overtakings "N" which is given at the bottom of the last column.

Equation 4.5.3-5 is then used to compute passenger car unit equivalencies at different speeds of the overtaken vehicles and are presented in the separate table following in sequence. Different flows were considered to measure passenger car unit equivalencies ranging from 730 VPH to 1,790 VPH. Summary tables (4.10.2-11) to (4.10.2-14) show vehicles type passenger car unit equivalencies at different flow levels. Each table is represented by a graph where speeds of slower vehicles are

No. of vehicles Q_1 at slower speed S_1 (Km/h)		Number of vehicles Q_2 at faster speed S_2 (Km/h)												Total
		-	1	2	8	20	64	46	16	2	-	-	Q_2	
Q_1	$\frac{1}{S_1}$	0.0083	0.0091	0.010	0.0111	0.0125	0.0143	0.0167	0.0200	0.0250	0.0333	0.050	$\frac{1}{S_2}$	
	S_1	120	110	100	90	80	70	60	50	40	30	20	S_2	
-	0.0500	0.0417	0.0409	0.0400	0.0390	0.0375	0.0357	0.0333	0.0300	0.0250	0.0167	-		
	20	-	-	-	-	-	-	-	-	-	-	-		
4	0.0333	0.0250	0.0242	0.0233	0.0222	0.0208	0.0190	0.0166	0.0133	0.0083	-			
	30	-	0.097	0.018	0.710	1.6664	4.864	3.054	0.851	0.066	-			11.493
8	0.0250	0.0167	0.0159	0.0150	0.0139	0.0125	0.0107	0.0083	0.005	-				
	40	-	0.080	0.150	0.556	1.250	3.424	1.909	0.40	-				7.769
16	0.0200	0.0117	0.0109	0.0100	0.0089	0.0075	0.0057	0.0033	-					
	50	-	0.1744	0.320	1.139	2.40	5.837	2.428	-					12.198
32	0.0167	0.0083	0.0076	0.0067	0.0056	0.0042	0.0024	-						
	60	-	0.243	0.429	1.434	2.688	4.915	-						9.7088
10	0.0143	0.0060	0.0052	0.0043	0.0032	0.0018	-							
	70	-	0.052	0.086	0.256	0.360	-							0.754
-	0.0175	0.0042	0.0034	0.0025	0.0014	-								
	80	-	-	-	-	-								
-	0.0111	0.0028	0.0020	0.0011	-									
	90	-			-									
-	0.0100	0.0017	0.0009	-										
	100	-		-										
-	0.0091	0.0008	-											
	110	-	-											
-	0.0083	-												
	120	-												
Total													Σ N	42.0225

Total flow (730 VPH)

1	0.0500	0.0417	0.0409	0.040	0.0390	0.0375	0.0357	0.0333	0.0300	0.0250	0.0167			
	20	-	0.041	0.08	0.312	0.750	2.285	1.532	0.480	0.05			-	5.53

Table 4.10.2-1 Matrix for Determining Passenger Car Equivalents (Cars and Light Goods Vehicles).

Total Flow (730 V/H)

Speed of Slower Vehicles (KM/H)	Passenger Car Unit Equivalents
20	13.16
30	6.83
40	2.31
50	1.83
60	1.00
70	1.00

Table 4.10.2-2 Passenger Car Unit Equivalents/ Speed of Slower Vehicles Relationship (Cars and Light Goods Vehicles).

No. of vehicles Q_1 at slower speed S_1 (Km/h)		Number of vehicles Q_2 at faster speed S_2 (Km/h)												Total
		1	2	3	4	11	22	27	13	-	-	-	Q_2	
Q_1	$\frac{1}{S_1}$ S_1	0.0083 120	0.0091 110	0.010 100	0.0111 90	0.0125 80	0.0143 70	0.0167 60	0.0200 50	0.0250 40	0.0333 30	0.050 20	$\frac{1}{S_2}$ S_2	
-	0.0500 20	0.0417 -	0.0409 -	0.0400 -	0.0390 -	0.0375 -	0.0357 -	0.0333 -	0.0300 -	0.0250 -	0.0167 -	- -		
4	0.0333 30	0.0250 0.100	0.0242 0.1936	0.0233 0.2796	0.0222 0.3552	0.0208 0.9152	0.0190 1.672	0.0166 1.7928	0.0133 0.6916	0.0083 -	- -			6.00
4	0.0250 40	0.0167 0.0668	0.0159 0.1272	0.0150 0.180	0.0139 0.2224	0.0125 0.550	0.0107 0.9416	0.0083 0.8964	0.005 0.260	- -				3.2444
18	0.0200 50	0.0117 0.2106	0.0109 0.3924	0.0100 0.540	0.0089 0.6408	0.0075 1.485	0.0057 2.2572	0.0033 1.6038	- -					7.1298
11	0.0167 60	0.0083 0.0913	0.0076 0.1671	0.0067 0.2211	0.0056 0.2464	0.0042 0.5082	0.0024 0.5808	- -						1.8150
3	0.0143 70	0.0060 0,0180	0.0052 0.0312	0.0043 0.0387	0.0032 0.0384	0.0018 0.0594	- -							0.18570
1	0.0175 80	0.0042 0.0042	0.0034 0.0068	0.0025 0.0075	0.0014 0.0056	- -								0.0241
-	0.0111 90	0.0028 -	0.0020 -	0.0011 -	- -									-
-	0.0100 100	0.0017 -	0.0009 -	- -										
-	0.0091 110	0.0008 -	- -											
-	0.0083 120	- -												
Total													ΣN	18.399

Total flow (1070 VPH)

1	0.0500 20	0.0417 0.0417	0.0409 0.0818	0.040 0.120	0.0390 0.1560	0.0375 0.4125	0.0357 0.7854	0.0333 0.8991	0.0300 0.390	0.0250	0.0167			2.89

Table 4.10.2-3 Matrix for Determining Passenger Car Equivalents (Cars and Light Goods Vehicles).

Total Flow (1070 V/H)

Speed of Slower Vehicles (KM/H)	Passenger Car Unit Equivalents
20	15.71
30	8.15
40	4.41
50	2.15
60	1.00
70	1.00

Table 4.10.2-4 Passenger Car Unit Equivalents/ Speed of Slower Vehicles Relationship (Cars and Light Goods Vehicles).

No. of vehicles Q_1 at slower speed S_1 (Km/h)		Number of vehicles Q_2 at faster speed S_2 (Km/h)												Total
		-	1	2	6	10	30	20	16	-	-	-	Q_2	
Q_1	$\frac{1}{S_1}$	0.0083	0.0091	0.010	0.0111	0.0125	0.0143	0.0167	0.0200	0.0250	0.0333	0.050	$\frac{1}{S_2}$	
	S_1	120	110	100	90	80	70	60	50	40	30	20	S_2	
-	0.0500	0.0417	0.0409	0.0400	0.0390	0.0375	0.0357	0.0333	0.0300	0.0250	0.0167	-		
	20	-	-	-	-	-	-	-	-	-	-	-		
2	0.0333	0.0250	0.0242	0.0233	0.0222	0.0208	0.0190	0.0166	0.0133	0.0083	-			
	30	-	0.0484	0.0932	0.2664	0.416	1.140	0.664	0.4256	-				3.0536
4	0.0250	0.0167	0.0159	0.0150	0.0139	0.0125	0.0107	0.0083	0.005	-				
	40	-	0.0636	0.120	0.3336	0.500	1.284	0.664	0.320	-				3.2852
13	0.0200	0.0117	0.0109	0.0100	0.0089	0.0075	0.0057	0.0033	-					
	50	-	0.1417	0.26	0.6942	0.975	2.223	0.8580	-					5.1519
22	0.0167	0.0083	0.0076	0.0067	0.0056	0.0042	0.0024	-						
	60	-	0.1672	0.2948	0.7392	0.924	1.584	-						3.7092
10	0.0143	0.0060	0.0052	0.0043	0.0032	0.0018	-							
	70	-	0.0520	0.0860	0.1920	0.1800	-							0.510
5	0.0175	0.0042	0.0034	0.0025	0.0014	-								
	80	-	0.0170	0.025	0.042	-								0.084
-	0.0111	0.0028	0.0020	0.0011	-									
	90	-	-	-	-									
-	0.0100	0.0017	0.0009	-										
	100	-	-	-										
-	0.0091	0.0008	-											
	110	-	-											
-	0.0083	-												
	120	-												
Total													Σ N	15.79

Total flow (1400 VPH)

1	0.0500	0.0417	0.0409	0.040	0.0390	0.0375	0.0357	0.0333	0.0300	0.0250	0.0167			
	20	-	0.0409	0.080	0.2340	0.375	1.071	0.666	0.48	-				2.95

Table 4.10.2-5 Matrix for Determining Passenger Car Equivalents (Cars and Light Goods Vehicles).

Total Flow (1400 V/H)

Speed of flow Vehicles (KM/H)	Passenger Car Unit Equivalents
20	18.68
30	9.66
40	5.20
50	2.51
60	1.07
70	1.00

Table 4.10.2-6 Passenger Car Unit Equivalents/ Speed of Slower Vehicles Relationship (Cars and Light Goods Vehicles).

No. of vehicles Q_1 at slower speed S_1 (Km/h)		Number of vehicles Q_2 at faster speed S_2 (Km/h)												Total
		-	1	4	6	10	29	34	13	3	-	-	Q_2	
Q_1	$\frac{1}{S_1}$	0.0083	0.0091	0.010	0.0111	0.0125	0.0143	0.0167	0.0200	0.0250	0.0333	0.050	$\frac{1}{S_2}$	
	S_1	120	110	100	90	80	70	60	50	40	30	20	S_2	
-	0.0500	0.0417	0.0409	0.0400	0.0390	0.0375	0.0357	0.0333	0.0300	0.0250	0.0167	-		
	20	-	-	-	-	-	-	-	-	-	-	-		
2	0.0333	0.0250	0.0242	0.0233	0.0222	0.0208	0.0190	0.0166	0.0133	0.0083	-			
	30	-	0.0484	0.1864	0.2664	0.416	0.102	0.1288	0.3458	0.0498	-			3.5460
5	0.0250	0.0167	0.0159	0.0150	0.0139	0.0125	0.0107	0.0083	0.005	-				
	40	-	0.0795	0.300	0.4170	0.625	1.5515	1.4110	0.325	-				4.7090
20	0.0200	0.0117	0.0109	0.0100	0.0089	0.0075	0.0057	0.0033	-					
	50	-	0.218	0.80	1.068	1.500	3.306	2.04	-					8.9320
9	0.0167	0.0083	0.0076	0.0067	0.0056	0.0042	0.0024	-						
	60	-	0.0684	0.2412	0.3024	0.3780	0.6264	-						1.61640
2	0.0143	0.0060	0.0052	0.0043	0.0032	0.0018	-							
	70	-	0.0104	0.0344	0.0384	0.036	-							0.11920
-	0.0175	0.0042	0.0034	0.0025	0.0014	-								
	80	-	-	-	-	-								
-	0.0111	0.0028	0.0020	0.0011	-									
	90	-	-	-	-									
-	0.0100	0.0017	0.0009	-										
	100	-	-	-										
-	0.0091	0.0008	-											
	110	-	-											
-	0.0083	-												
	120	-												
Total													Σ N	18.920

Total flow (1730 VPH)

1	0.0500	0.0417	0.0409	0.040	0.0390	0.0375	0.0357	0.0333	0.0300	0.0250	0.0167			
	20	-	0.040	0.1560	0.225	0.357	1.0353	1.1322	0.390	0.0750				3.411

Table4.10.2-7 Matrix for Determining Passenger Car Equivalents (Cars and Heavy Goods Vehicles).

Total Flow 1730 V/H)

Speed of Slower Vehicles (KM/H)	Passenger Car Unit Equivalents
20	18.03
30	9.37
40	4.98
50	2.36
60	1.00
70	1.00

Table 4.10.2-8 Passenger Car Unit Equivalents/ Speed of Slower Vehicles Relationship (Cars and Heavy Goods Vehicles).

No. of vehicles Q_1 at slower speed S_1 (Km/h)		Number of vehicles Q_2 at faster speed S_2 (Km/h)												Total
		1	1	9	22	30	27	9	1	-	-	-	Q_2	
Q_1	$\frac{1}{S_1}$ S_1	0.0083 120	0.0091 110	0.010 100	0.0111 90	0.0125 80	0.0143 70	0.0167 60	0.0200 50	0.0250 40	0.0333 30	0.050 20	$\frac{1}{S_2}$ S_2	
-	0.0500 20	0.0417 -	0.0409 -	0.0400 -	0.0390 -	0.0375 -	0.0357 -	0.0333 -	0.0300 -	0.0250 -	0.0167 -	- -		
-	0.0333 30	0.0250 -	0.0242 -	0.0233 -	0.0222 -	0.0208 -	0.0190 -	0.0166 -	0.0133 -	0.0083 -	- -			
2	0.0250 40	0.0167 0.0334	0.0159 0.0318	0.0150 0.270	0.0139 0.6116	0.0125 0.750	0.0107 0.5778	0.0083 0.1494	0.005 0.010	- -				2.4336
6	0.0200 50	0.0117 0.0702	0.0109 0.0654	0.0100 0.540	0.0089 1.1748	0.0075 0.350	0.0057 0.9234	0.0033 0.1620	- -					4.2858
8	0.0167 60	0.0083 0.0664	0.0076 0.0608	0.0067 0.4824	0.0056 0.9856	0.0042 1.008	0.0024 0.5184	- -						3.1216
20	0.0143 70	0.0060 0.120	0.0052 0.104	0.0043 0.774	0.0032 1.408	0.0018 1.080	- -							3.486
3	0.0175 80	0.0042 0.0126	0.0034 0.0102	0.0025 0.0675	0.0014 0.0924	- -								0.1827
-	0.0111 90	0.0028 -	0.0020 -	0.0011 -	- -									
	0.0100 100	0.0017	0.0009	- -										
	0.0091 110	0.0008 -	-											
	0.0083 120	- -												
Total													Σ N	13.510

Total flow (1730 VPH)

1	0.0500 20	0.0417 0.0417	0.0409 0.0409	0.040 0.360	0.0390 0.858	0.0375 1.1250	0.0357 0.9639	0.0333 0.2997	0.0300 0.030	0.0250 -	0.0167			3.72

Table 4.10.2-9 Matrix for Determining Passenger Car Equivalents (Cars and Articulated Goods Vehicles).

Total Flow (1730 V/H)

Speed of Slower Vehicles (KM/H)	Passenger Car Unit Equivalents
20	27.54
30	15.15
40	9.01
50	5.29
60	2.89
70	1.16

Table 4.10.2-10 Passenger Car Unit Equivalents/ Speed of Slower Vehicles Relationship (Cars and Articulated Goods Vehicles).

Speed of Slower Vehicle (KM/H)	Passenger Car Unit Equivalences		
	Light Goods Vehicle	Heavy Goods Vehicle	Articulated Goods Vehicle
20	13.16	21.10	33.30
30	6.83	10.70	18.43
40	2.31	5.60	10.96
50	1.83	2.62	6.50
60	1.00	1.10	3.513
70	1.00	1.00	1.50

Table 4.1 0.2-11 Passenger Car Unit Equivalences/Speed of Slower Vehicle Relationship (Total Flow 730 V/H).

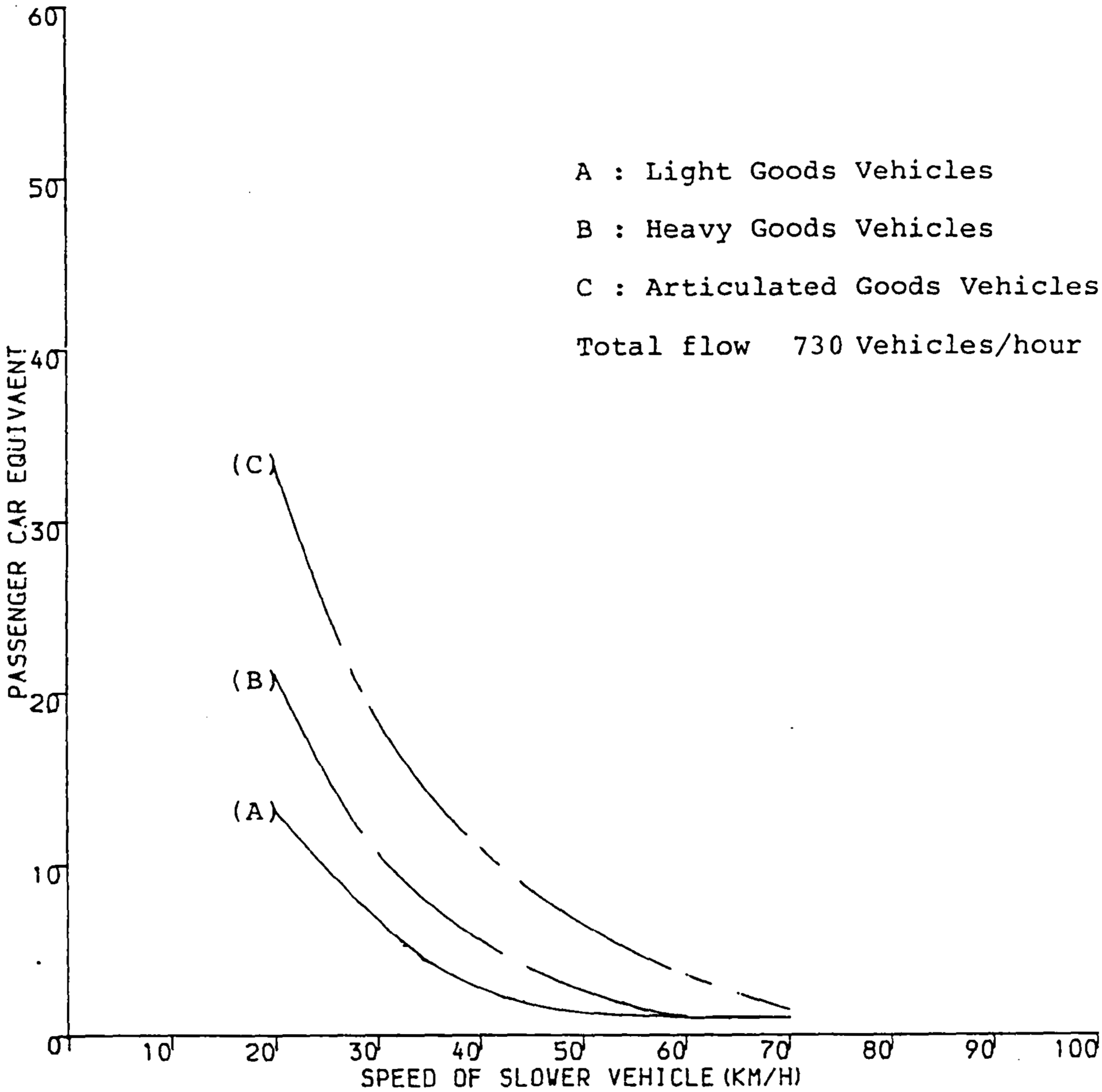


Figure 4.10.2-1 Speed of Slower Vehicle/Passenger Car Equivalent Relationship.

Speed of Slower Vehicle (KM/H)	Passenger Car Unit Equivalences		
	Light Goods Vehicle	Heavy Goods Vehicle	Articulated Goods Vehicle
20	18.60	24.10	38.42
30	9.66	13.60	21.36
40	5.20	7.82	12.00
50	2.51	3.90	6.68
60	1.07	1.30	3.80
70	1.00	1.00	1.50

Table 4.10.2-12 Passenger Car Unit Equivalences/Speed of Slower Vehicle Relationship (Total Flow 1400 V/H).

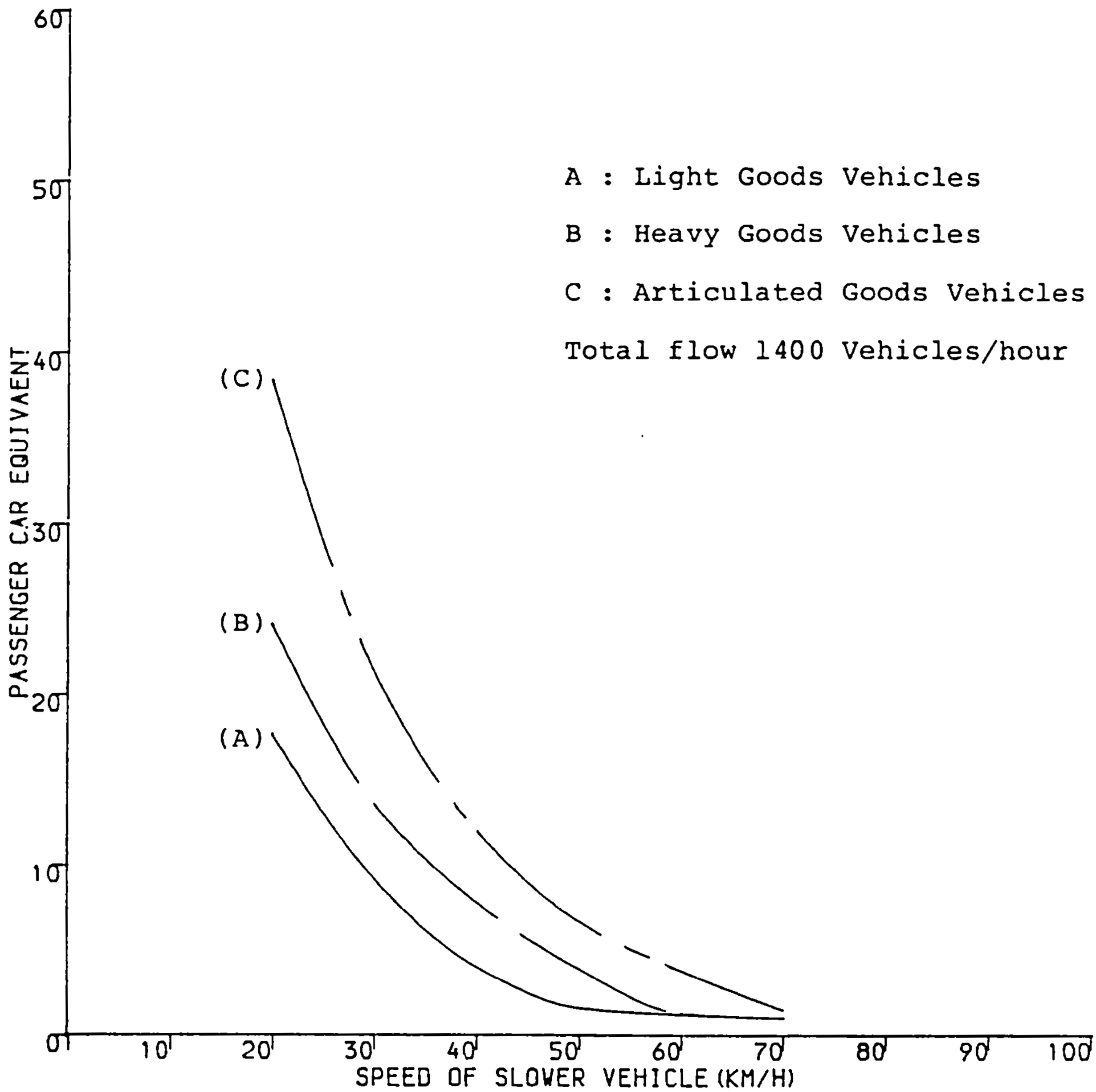


Figure 4.10.2-2 Speed of Slower Vehicle/Passenger Car Equivalent Relationship.

Speed of Slower Vehicle (KM/H)	Passenger Car Unit Equivalences		
	Light Goods Vehicle	Heavy Goods Vehicle	Articulated Goods Vehicle
20	15.71	22.30	35.71
30	8.15	12.40	24.20
40	4.41	7.13	15.36
50	2.15	3.34	8.40
60	1.00	1.22	3.80
70	1.00	1.00	1.52

Table 4.10.2-13 Passenger Car Unit Equivalences/Speed of Slower Vehicle Relationship (Total Flow 1070 V/H).

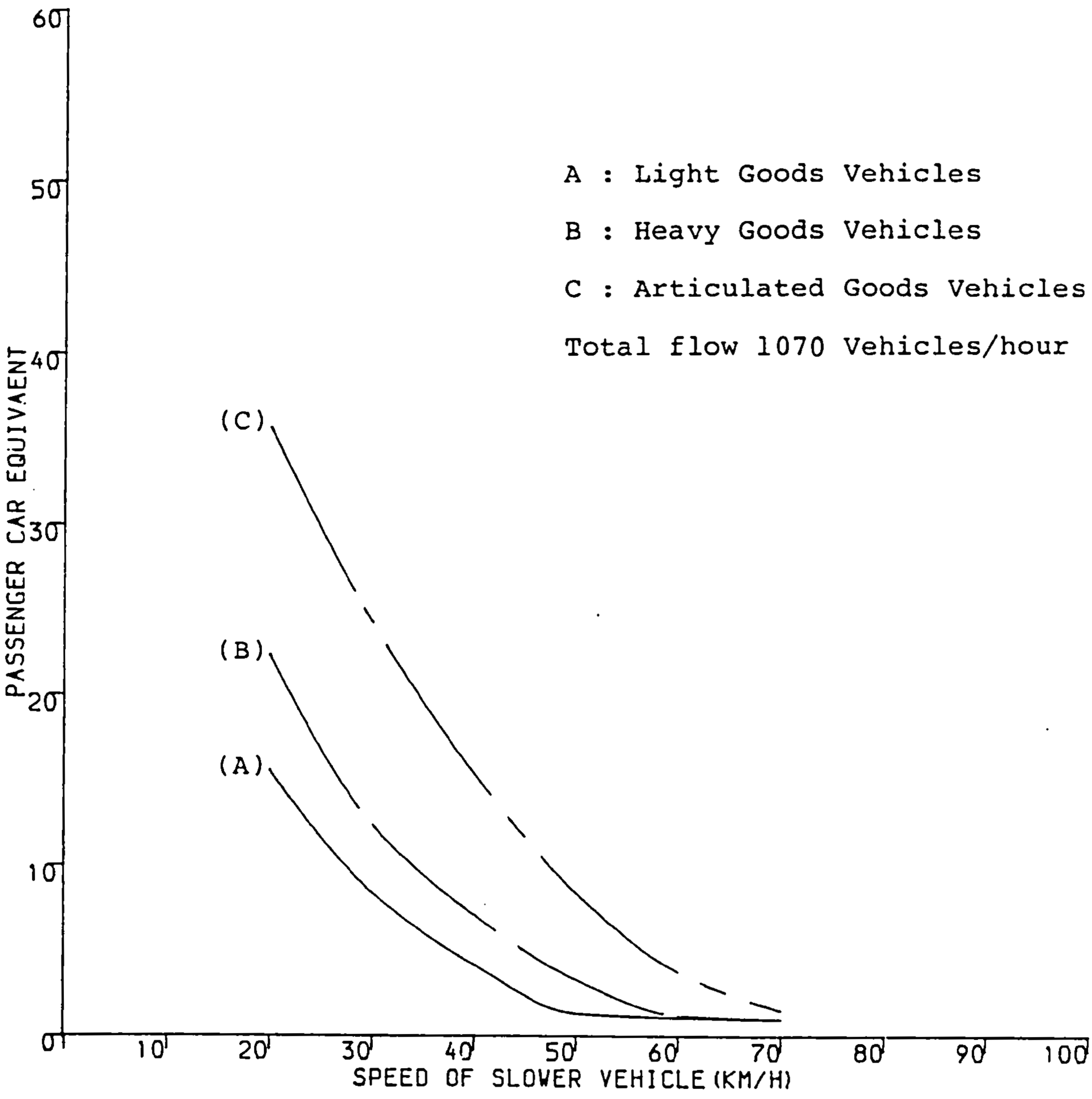


Figure 4.10.2-3 Speed of Slower Vehicle/Passenger Car Equivalent Relationship.

Speed of Slower Vehicle (KM/H)	Passenger Car Unit Equivalences		
	Light Goods Vehicle	Heavy Goods Vehicle	Articulated Goods Vehicle
20	8.80	18.03	27.54
30	4.30	9.37	15.15
40	2.20	4.98	9.01
50	1.10	2.36	5.29
60	1.00	1.00	2.89
70	1.00	1.00	1.16

Table 4.10.2-14 Passenger Car Unit Equivalences/Speed of Slower Vehicle Relationship (Total Flow 1790 V/H).

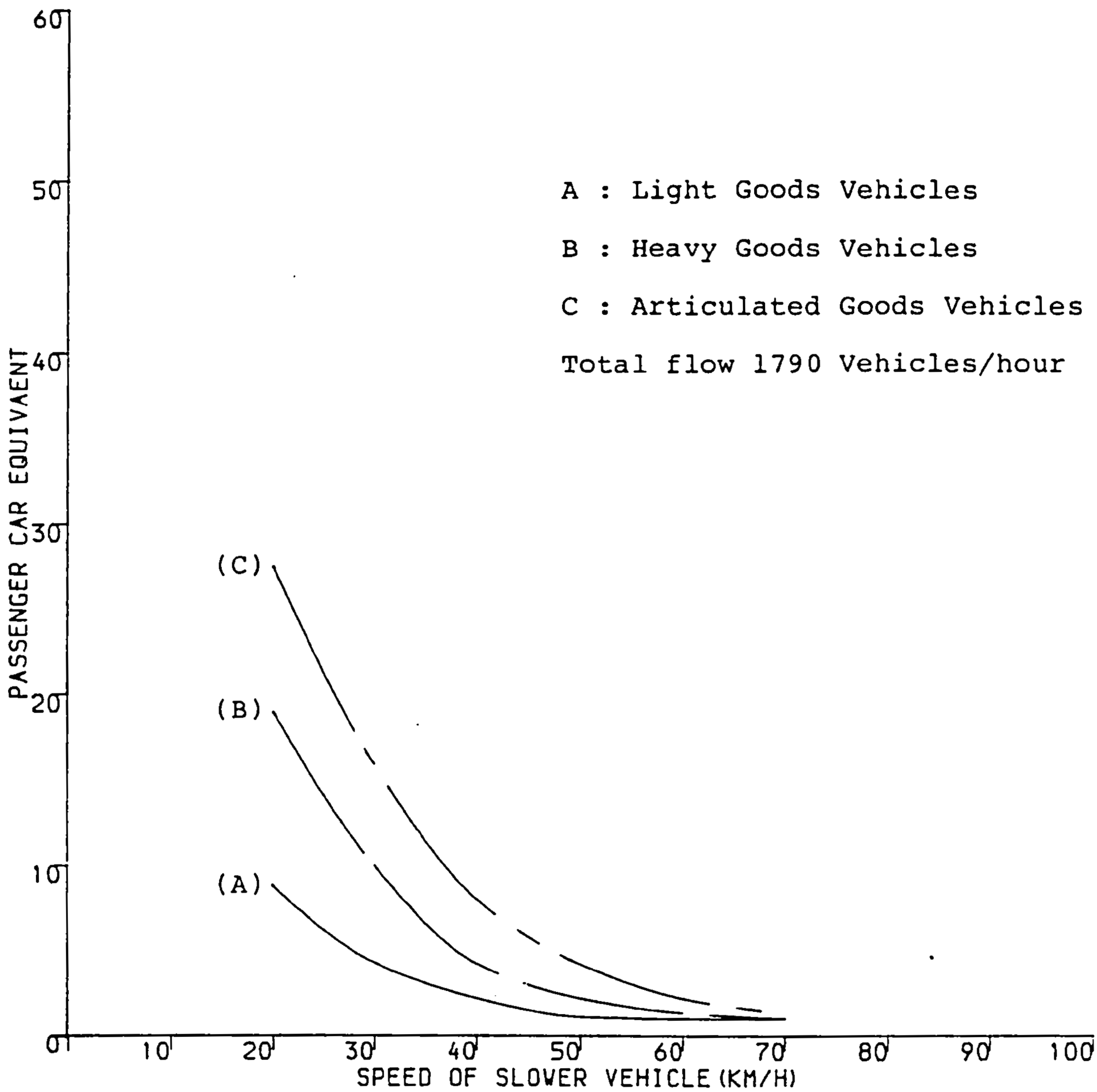


Figure 4.10.2-4 Speed of Slower Vehicle/Passenger Car Equivalent Relationship.

plotted versus their corresponding passenger car unit equivalents.

Figures (4.10.2-1) to (4.10.2-4) show that passenger car unit equivalents increase as the speed of slower vehicles decrease. This increase becomes greater at low speeds between 30 to 20 km/h.

The values of passenger car unit equivalents at different levels of flows show a steady increase in value between flows of 730 VPH to approm.14.00 VPH per lane. The PCU values become lower at flows over 1,500 VPH per lane as shown in figure (4.10.2-4). This is because the flow approaches lane capacity which affect speeds and consequently reduces overtaking procedures.

The passenger car unit equivalents computed indicate the effect of vehicle types on traffic flow performance by occupying more space, reducing other vehicles' opportunities to travel at higher speeds and restricting other vehicles' abilities to overtake, especially with higher percentages of heavy commercial vehicles.

From the results of the analysis of overtaking or passing behaviour the effects of different vehicle types were established by computing passenger car unit equivalencies.

At the 10th, 50th and 90th speed percentiles for each vehicle type, a passenger car unit value was obtained and represented by table (4.10.2-15). Equivalents for light goods vehicles show lower values than those of heavy and articulated goods vehicles, values of 3.10, 1.02, 1.00 were obtained at 10%, 50% and 90% of speed respectively. For heavy goods vehicles values of 5.97, 1.85, 1.00 were obtained at 10%, 50% and 90% of speed respectively. For articulated goods vehicles, PCU values were much higher, ranging from 21.58, 11.10 to 3.34 at 10%, 50% and 90% of speeds respectively. The PCU values indicated the difference in vehicle type characteristics. These vehicles are more likely to have significant effects on highway traffic flow performance, by reducing the ability of other vehicles to travel at their own desired speeds, by decreasing opportunities to overtake, and increasing the time and distance of overtaking procedure because of their enormous length. Thus highway capacity is reduced.

Vehicle type		Percentile speeds (km/h) 10%	50%	90%
Light goods vehicle	Speed	45.61	61.40	74.87
	PCU's	3.10	1.02	1.00
Heavy goods vehicle	Speed	36.34	51.46	69.02
	PCU's	5.97	1.85	1.00
Articulated good vehicle	Speed	27.33	42.66	62.88
	PCU's	21.58	11.10	3.34

Table 4.10.2-15 Average speed passenger car unit equivalents.

4.11 - An analysis of vehicle type effect on highway traffic parameters

4.11-1 - Effect of vehicle type on speed/flow/capacity relationships

Much research has been carried out into the relationships between speed, flow and density, at different levels of traffic flow and on various types of highway. Comparatively little research has been done with regard to the effect of vehicle type on speed/flow relationships.

This study investigates the changes in passenger car characteristics and other vehicle types, and how these changes have affected speed/flow relationships on a section of two-lane motorway under free-flow conditions.

The data used to investigate vehicle type effect were extracted from film analysis during the investigation of vehicle type speed study. They were recorded during weekday morning and afternoon peak periods on a two-lane motorway. The data refer to near side and far side lanes, and each of the 140 data points represents the mean speed of traffic flow over 10 minutes periods. Speed ranges from 35 to 120 km/h. In each set of observations, percentages of heavy commercial vehicles were noted at different traffic volumes. Both heavy and articulated goods vehicles were combined together

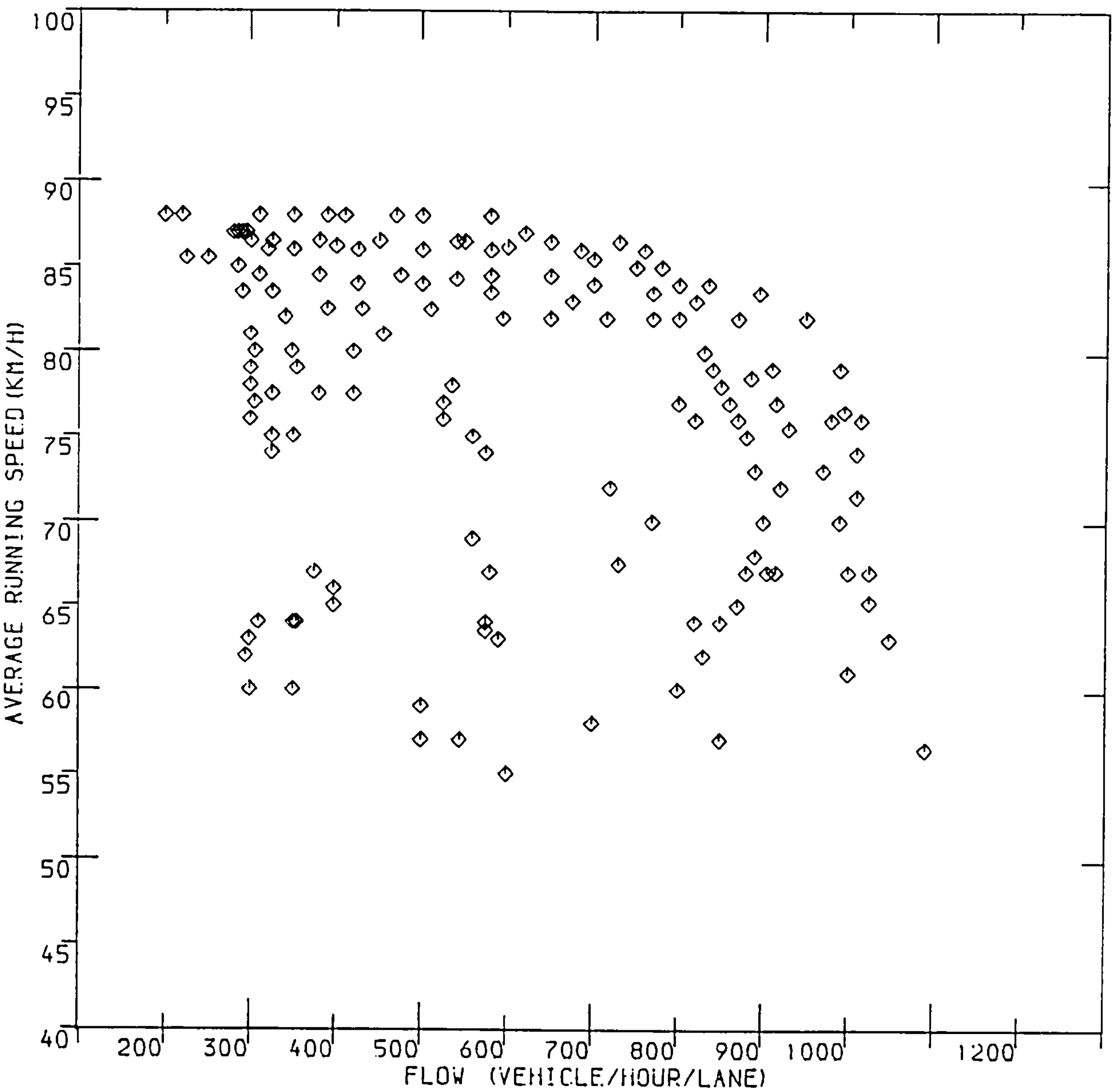


Figure 4.11.1-1 Observed average highway speed versus one lane flow (near-side lane) with 15 to 19%heavy commercial vehicles.

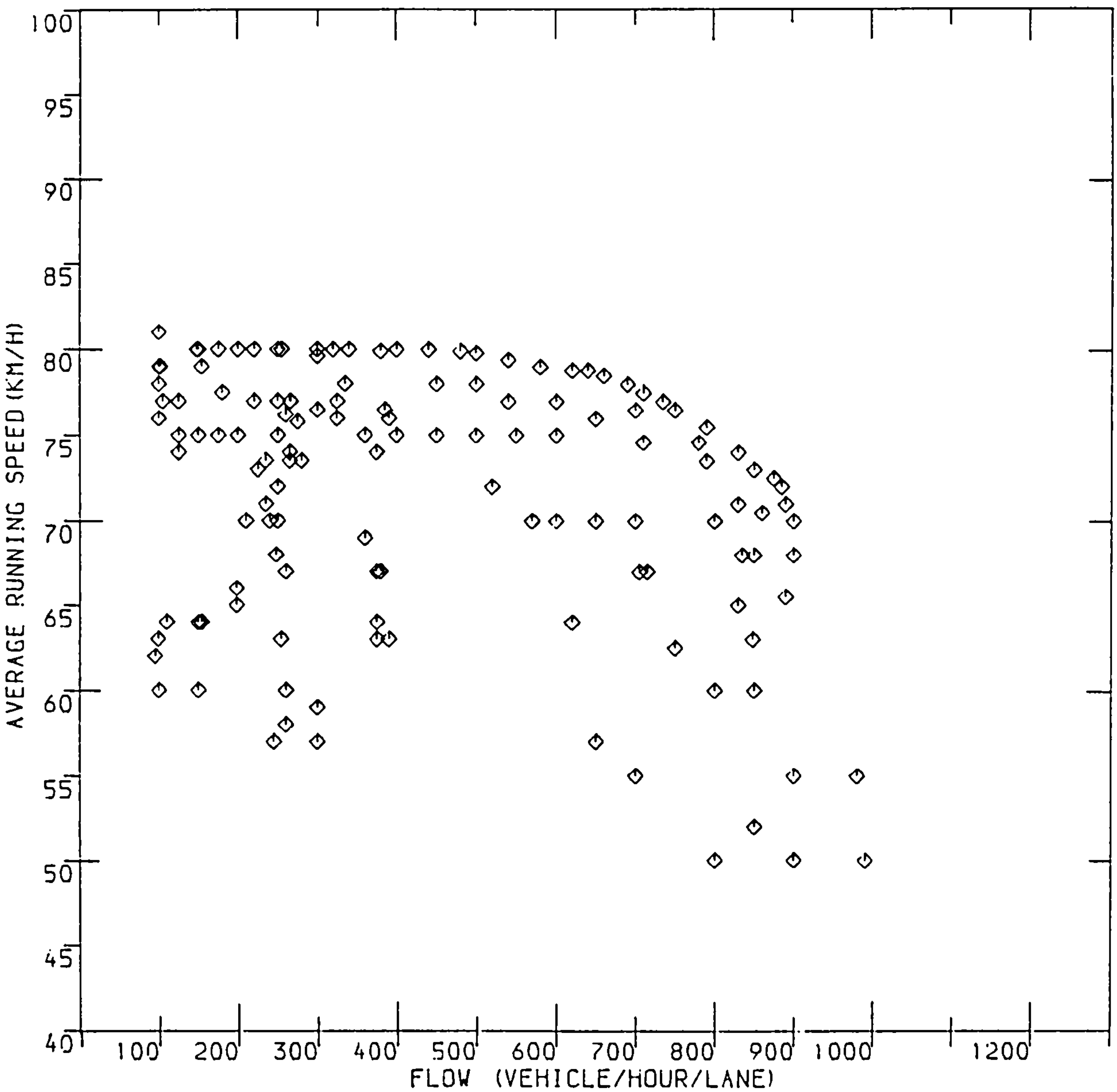


Figure 4.11.1-2 Observed average highway speed versus one lane flow (near-side lane) with 20 to 28%heavy commercial vehicles.

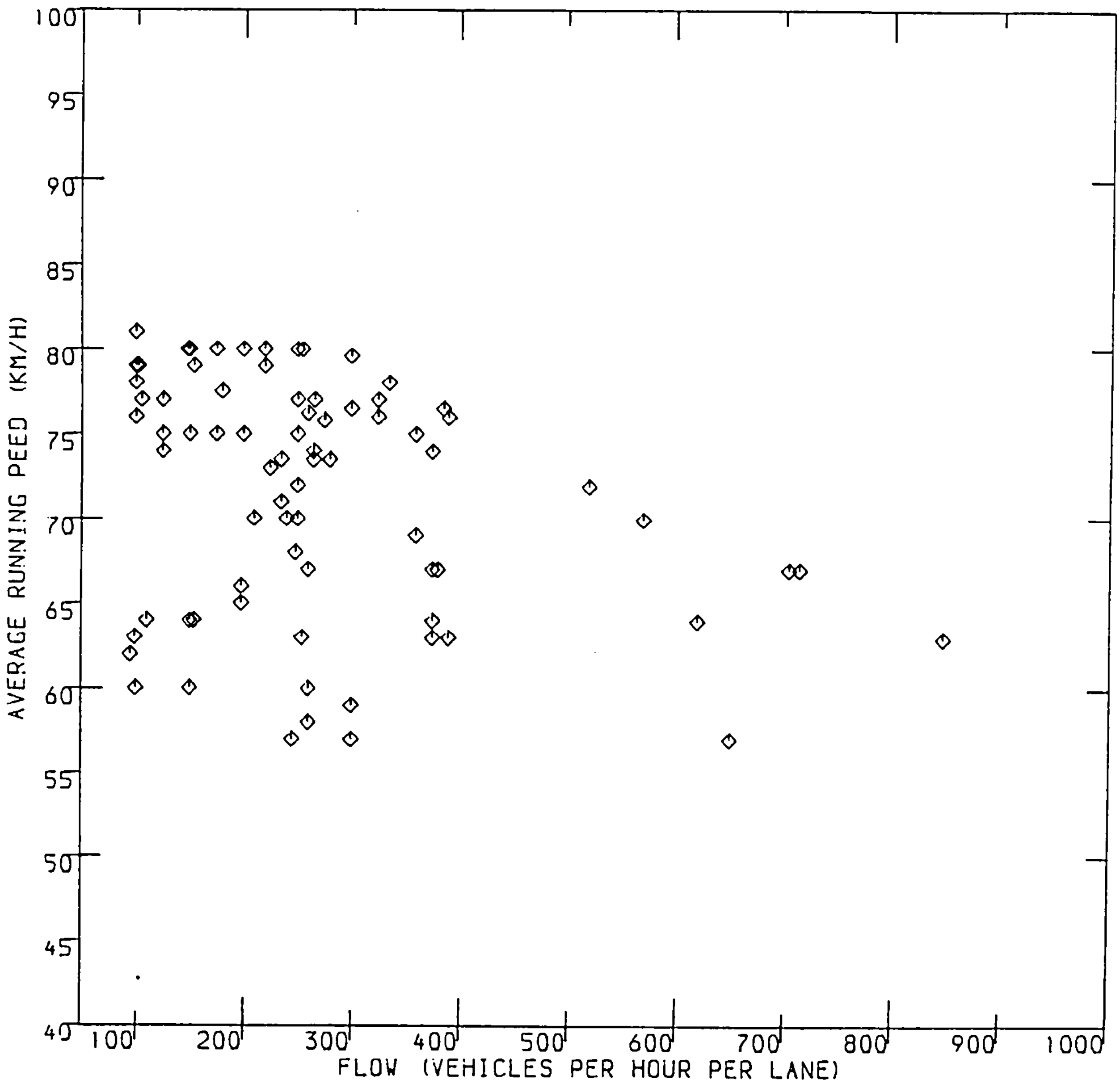


Figure 4.11.1-3 Observed average highway speed versus one lane flow (near-side lane) with 30 to 35% heavy commercial vehicles.

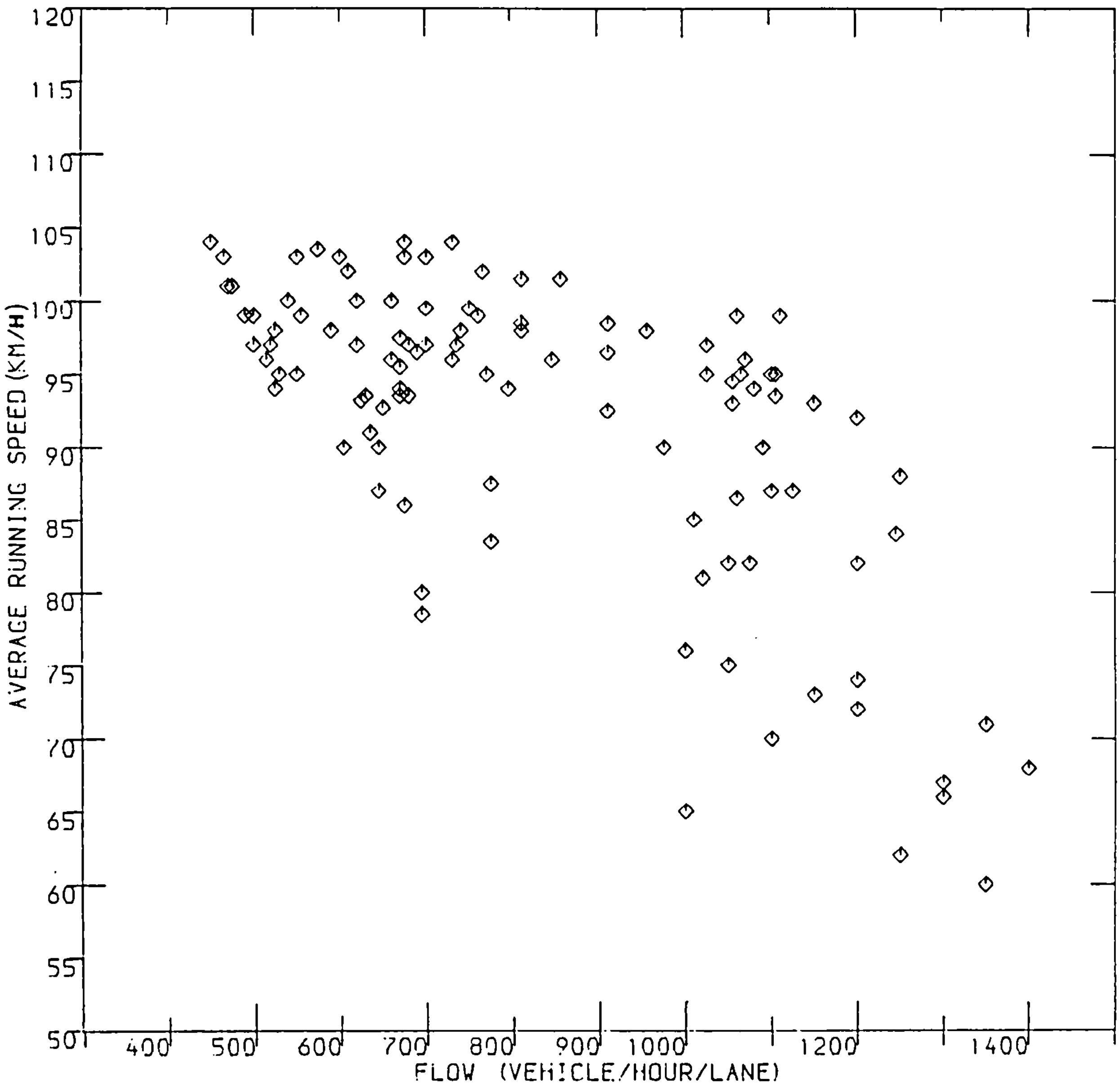


Figure 4.11.1-4 Observed average highway speed versus one lane flow (far-side lane) with 5 to 9%heavy commercial vehicles.

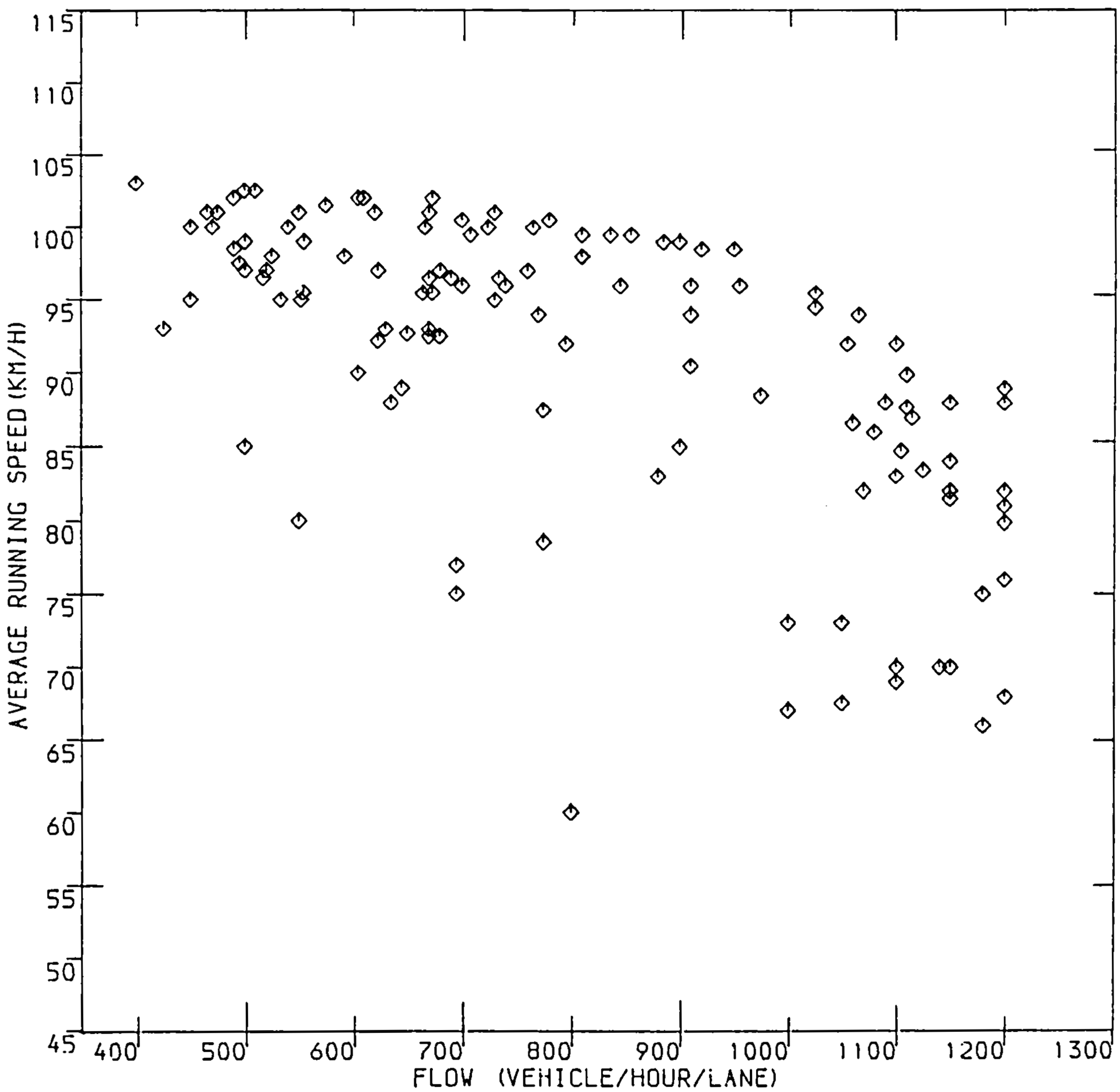


Figure 4.11.1-5 Observed average highway speed versus one lane flow (far-side lane) with 10 to 12% heavy commercial vehicles.

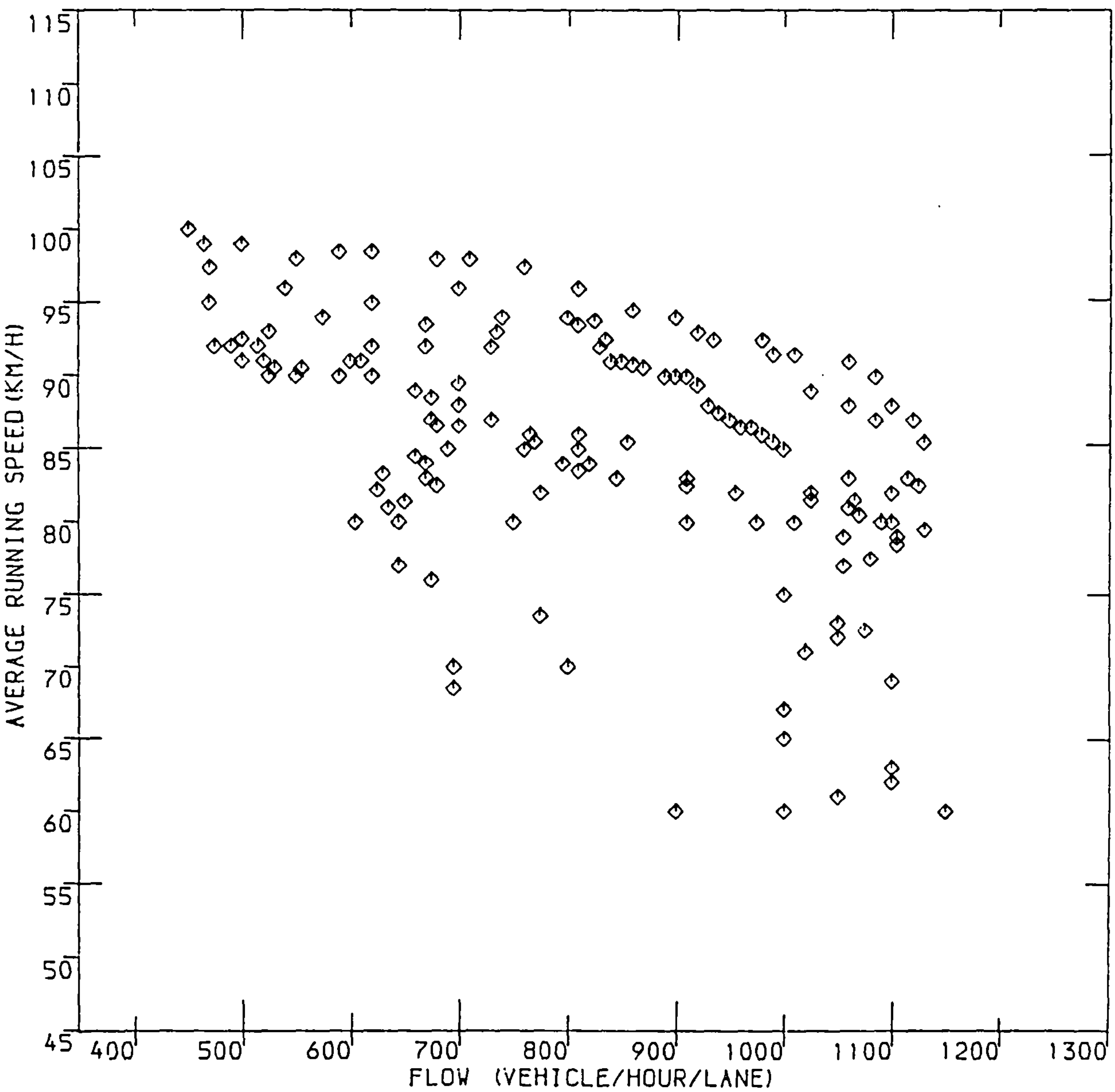


Figure 4.11.1-6 Observed average highway speed versus one lane flow (far-side lane) with12 to 16% heavy commercial vehicles.

The relationship of average running speeds to observed flows at various percentages of heavy commercial vehicles for two-lane motorway are presented by plotting the data obtained from the analysis. Observed flows are plotted versus average running speed as shown in figures (4.11.1-1) to (4.1 .1-6). Figures (4.11.1-1), (4.11.1-2) and (4.11.1-3) show the variation in speeds with the increase of traffic flow on the near side lane. Figures (4.11.1-4), (4.11.1-5) and (4.11.1-6) show the variation in speeds with the increase of traffic flow on the far side lane.

Despite the considerable scatter of the data plotted in the above mentioned figures, the scatter points do show the expected fall-off in average traffic flow speeds with the increase in traffic volumes per lane of travel.

Speed/flow relationship for the near side lane illustrated in figures (4.11.1-1) to (4.11.1-3) show lower speed values and decrease of speed with the increase of flows even at a low level of traffic volume where speeds fall sharply at around 1,100 vph, 950 vph and 700 vph. This is mainly due to high percentages of light, heavy and articulated goods vehicles. These types of vehicle occupy the near side lane where they normally travel at a lower speed than passenger cars. This also effects the maximum in one category as heavy commercial vehicles to investigate their effects on the speed/flow relationship. Because the purpose of this analysis is to identify the basic effects of vehicle types, only data representing ideal conditions were considered (i.e. no incidents, accidents or adverse weather).

Traffic on the M621 Motorway is heavy during peak periods, morning between 7.30 and 9.30 a.m., and afternoon between 4.00 and 6.00 p.m. Vehicle type average percentages during observations over a period of three months are as follows;

for near side lane	
heavy commercial vehicles	16% to 35%
light goods vehicles	10% to 18%
cars	47% to 74%
for far side lane	
heavy commercial vehicles	6% to 16%
light goods vehicles	12% to 23%
cars	65% to 82%

The data was maintained and analysed on a lane-by-lane basis because there were considerable differences in vehicle type percentages per lane of travel.

traffic volume that can pass per hour which results in reduced lane capacity compared with the Highway Capacity Manual(103), of 2,000 vehicles per hour per lane. The reduction in lane capacity is higher as the percentage of heavy commercial vehicles increase.

Figure (4.11.1-1) shows the fall-off is speeds at around 1,100 vphpl with 15 to 19 per cent of heavy commercial vehicles. Figures (4.11.1-2) and (4.11.1-3) show the fall-off in speeds at around 950 vphpl with 20 to 26 per cent of heavy commercial vehicles at around 700 vphpl with 30 to 35 per cent of heavy commercial vehicles respectively. The far side lane speed/flow relationship shows a different picture than that of the near side lane. Figures (4.11.1-4), (4.11.1-5) and (4.11.1-6) show a fall-off in traffic stream speeds at around 1,350, 1,200 and 1,100 vphpl with 6 to 9, 10 to 12 and 12 to 16 per cent of heavy commercial vehicles respectively. This clearly indicates that with lower percentages of heavy commercial vehicles in the traffic stream, higher values of speeds can be maintained and consequently higher traffic volumes can be accommodated per lane per hour which will result in the increase in capacity of the roadway.

A speed/flow curve can be fitted to the data by eye, showing the typical parabolic curve that was first introduced by the Highway Capacity Manual (103). This curve explains the relationship between vehicular average speeds and the flow accommodated by highways. Vehicle type effects on speed/flow relationship vary with total traffic composition and location of vehicle type in lane of travel. Also vehicle type has significant effects on speed/flow relationship on roadways with gradients where heavy vehicles travel at much lower speeds than on a level roadway.

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4.11.2 - Overtaking procedure and vehicle type effect

Overtaking and passing manoeuvres are carried out by drivers in order to maintain pace and they are required to make complex decisions regarding roadway, environmental and vehicular characteristics. Passing and overtaking opportunities are severely affected by slow moving heavy commercial vehicles on two and three-lane motorways, which have a significant effect on average running speeds of total traffic flow and consequently on the capacity of roads.

The phenomenon of passing and overtaking has been investigated to study the effect of large vehicles on traffic speed-flow relationships and its implications for capacity.

Passing and overtaking procedure was found to be based on the following:

(1) speed of the lead vehicle (vehicle to be overtaken),
(2) characteristics (capabilities) of the overtaking vehicle,
(3) the time or distance required to perform the manoeuvre,
(4) driver's estimate and decision,
(5) size (length and width) of the overtaken vehicle,
(6) gradient,
(7) percentages of heavy commercial vehicles,
(8) sight distance.

Large size commercial vehicles have an increased effect on other motorists' driving conditions resulting in blockage of road signs and inability to see forward which forces the drivers of small vehicles to reduce their speed and ability to overtake. Often the drivers of small cars find the view of the road ahead is suddenly obliterated by an articulated goods vehicle pullign into the lane in front of them, another one travelling behind while a third is passing and overtaking. This causes confusion, the missing of important directional or advisory signs and exits, and a reduction in speed. Another important effect of heavy commercial vehicles on the drivers of smaller cars is cutting in, when the driver of a heavy commercial vehicle decides to change lane after overtaking a small or large vehicle, where in some cases wrong estimates of when to cut-in are made due to the enormous length of their vehicles (12 - 15.40m). Such behaviour results in the reduction of speed of overtaken vehicles. These situation are well explained in figures (4.11.2-1) to (4.11.2-3).

Behaviour of individual vehicle types, distance and time required while overtaking have been investigated as they are an important traffic criterion affecting traffic flow, speed and road capacity. Numbers of overtakings and the distance required by each vehicle type to overtake other vehicles have been recorded.

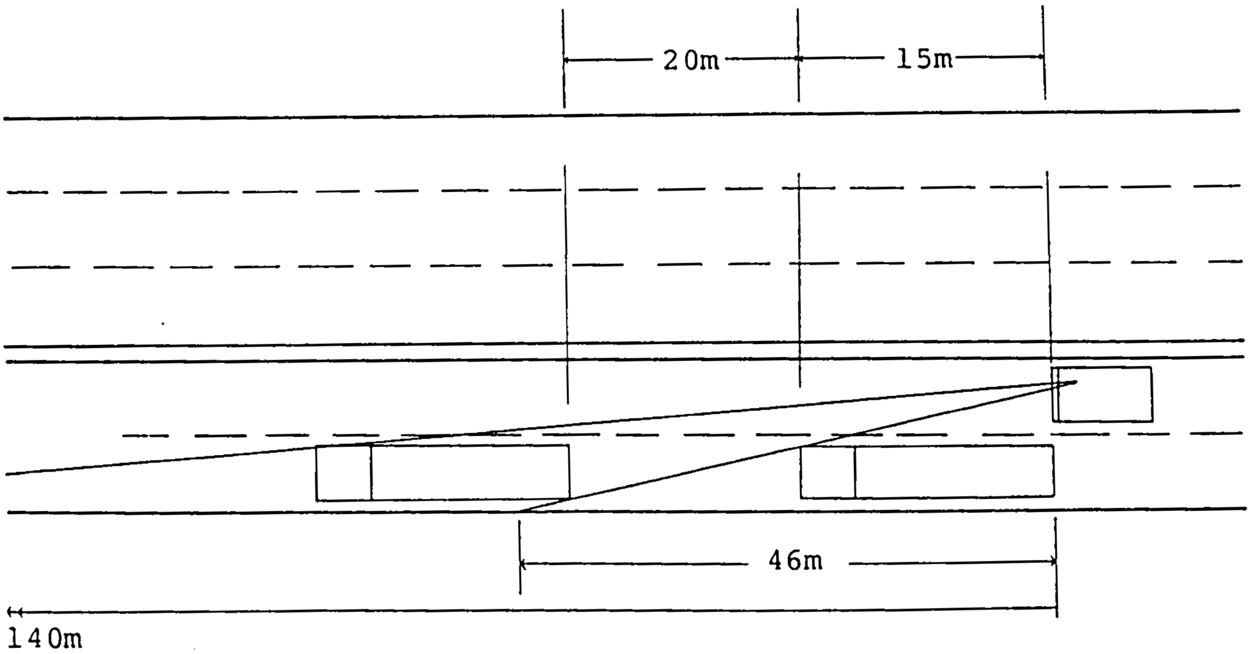

Figure 4.11.2-1 Horizontal sight line - passing mode.

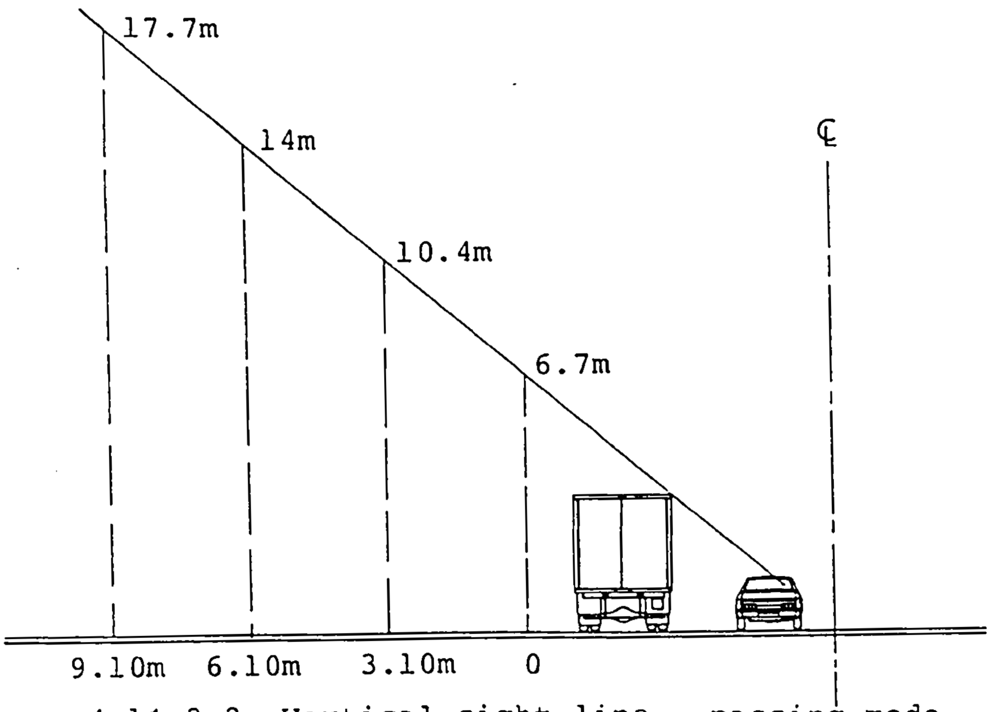

Figure 4.11.2-2 Vertical sight line - passing mode.

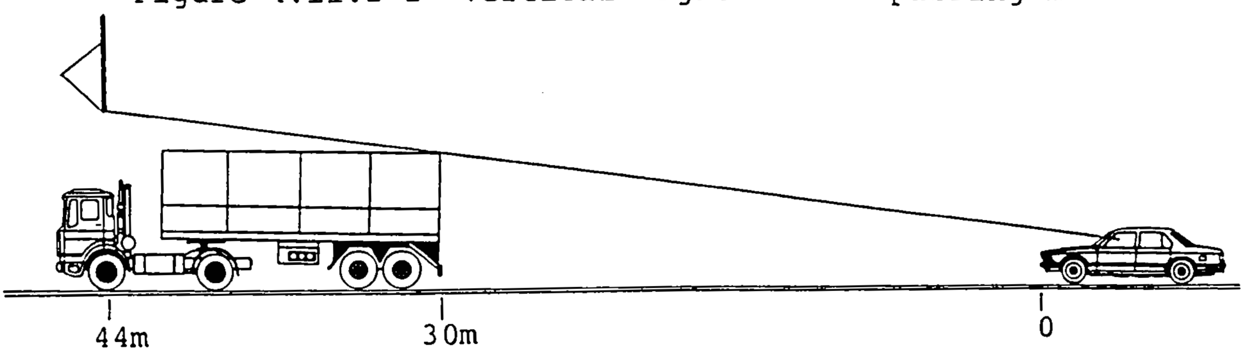

Figure 4.11.2-3 Vertical sight line - following mode.

Tables (4.11.2-1) to (4.11.2-4) show values recorded for passenger cars, light goods vehicles, heavy goods vehicles and articulated goods vehicles respectively. Also speeds of overtaking vehicles and overtaken vehicles were noted. Numbers of observed overtakings were plotted against distances required for overtaking and are shown in figures (4.11.2-4) to (4.11.2-7). Also speeds of overtaking for different vehicle types were plotted versus distance of overtaking as shown in figure (4.11.2-8). This figure shows that the distance required for overtaking increases as the vehicle size increases and decreases when the speed of overtaking vehicle increased. Also it was noted that the distance of overtakings increase sharply when the speed of vehicle to be overtaken exceeds 40 km/h. The data obtained from the relationship of the distance required for passenger cars to overtake other vehicles travelling at different speeds are shown in figure (4.11.2-9).

Vehicle types vary in their ability while performing overtaking procedures in terms of the distance and time they require at different speeds. For example a passenger car travelling at an average speed of 90 km/h requires an average distance of 75m to overtake. Therefore, the time taken by a passenger car to overtake is 7 seconds which means that the lane occupancy by a car while over-

Passing distance Class (m)	Observed No. of overtaking
50 - 69.9	2
70 - 99.9	6
100 - 119.9	10
120 - 139.9	12
140 - 159.9	4

Table 4.12.2-1 Observed number of passenger cars overtaking slow moving vehicles.

Passing distance Class (m)	Observed No. of overtaking
120 - 139.9	6
140 - 159.9	8
160 - 179.9	14
180 - 199.9	10

Table 4.12.2-2 Observed number of light goods vehicles overtaking slow moving vehicles.

Passing distance Class (m)	Observed No. of overtaking
140 - 159.9	4
160 - 199.9	8
170 - 199.9	12
200 - 219.9	10

Table 4.12.2-3 Observed number of heavy goods vehicles overtaking slow moving vehicles.

Passing distance Class (m)	Observed No. of overtaking
160 - 179.9	3
180 - 199.9	6
200 - 219.9	11
220 - 239.9	14
240 - 259.9	6

Table 4.12.2-4 Observed number of articulated goods vehicles overtaking slow moving vehicles.

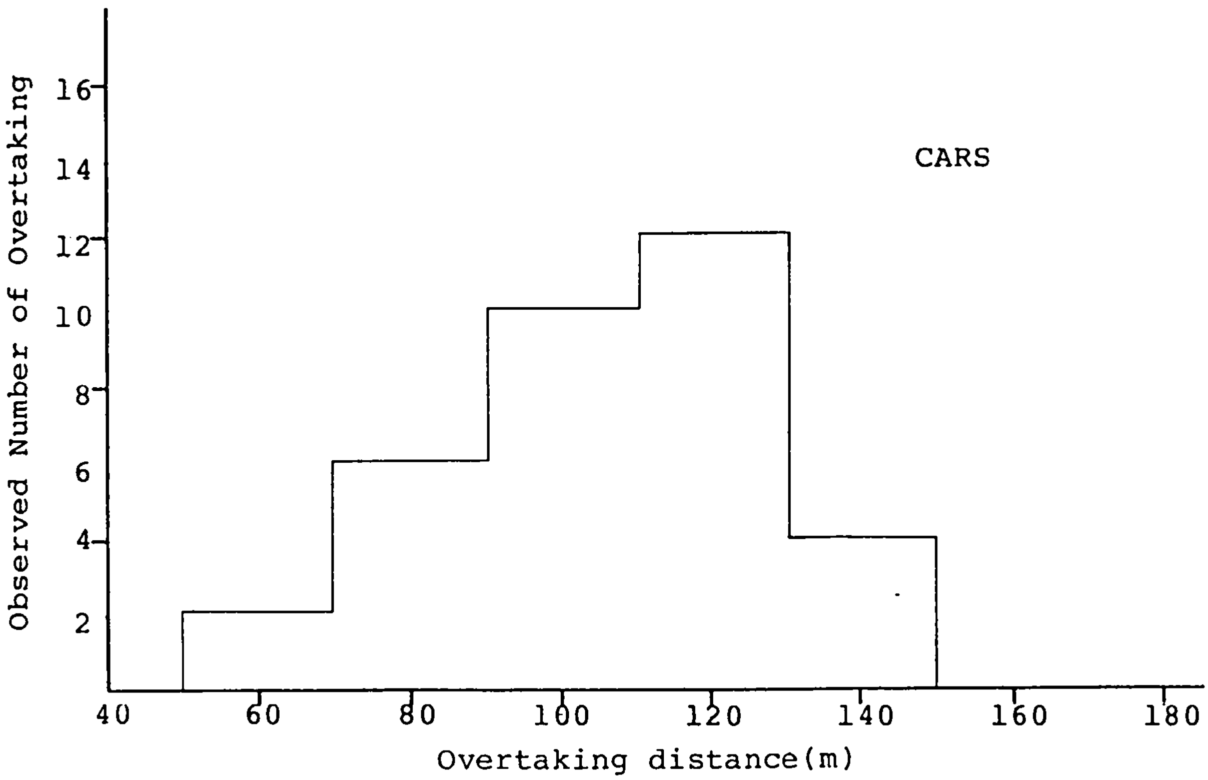

Figure 4.12.2-4 Distances of passenger cars overtaking slow moving vehicles.

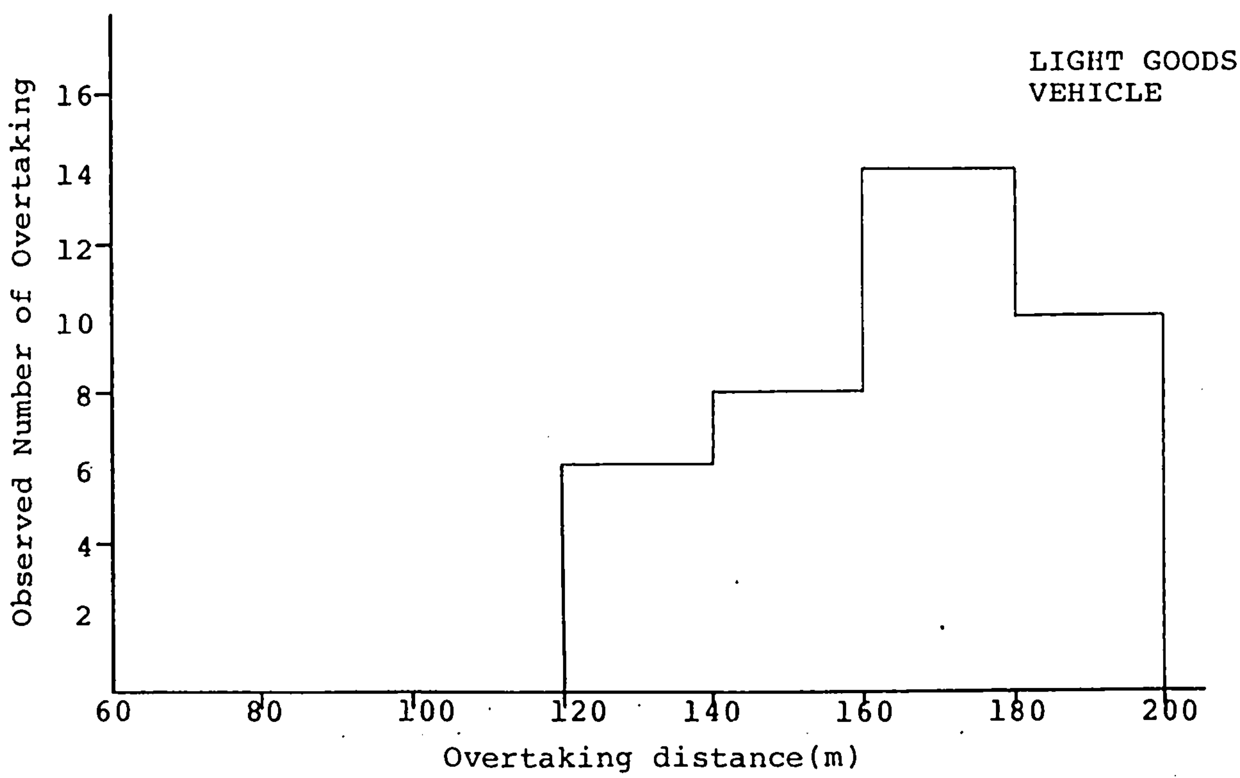

Figure 4.12.2-5 Distances of light goods vehicles overtaking slow moving vehicles.

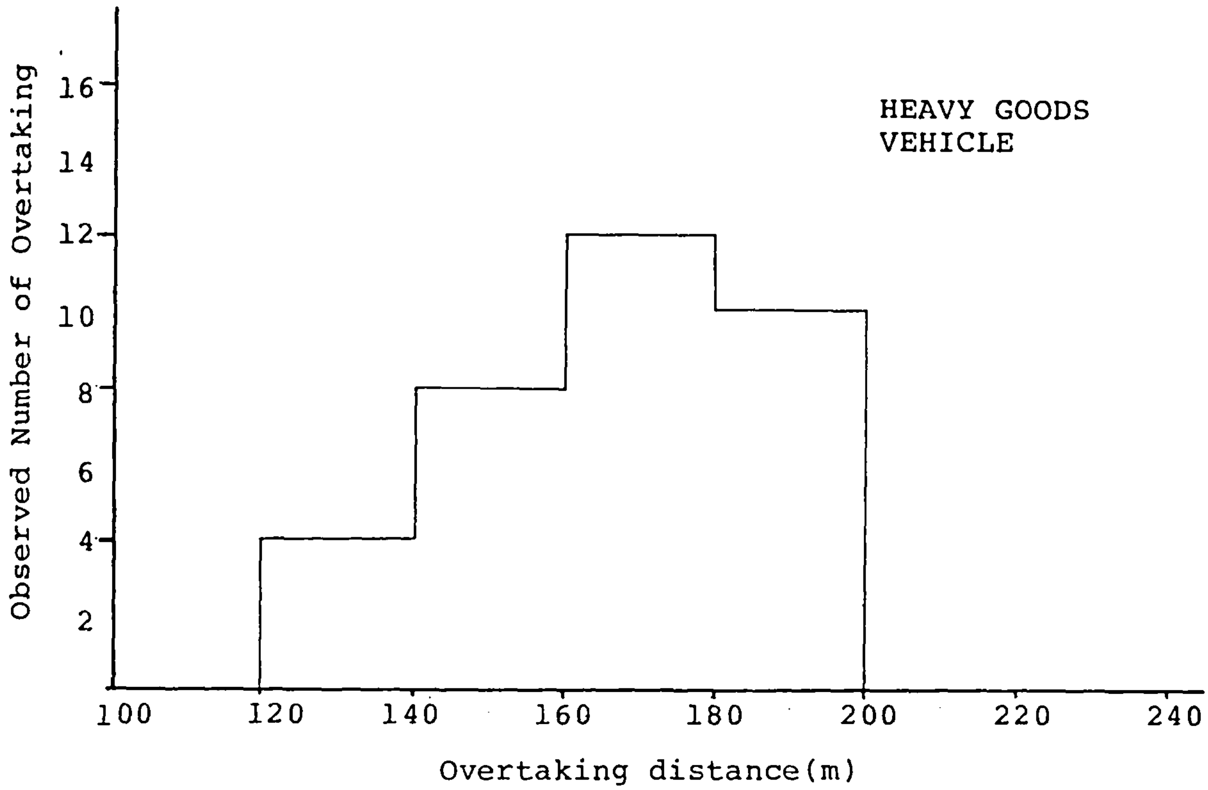

Figure 4.12.2-6 Distances of heavy goods vehicles overtaking slow moving vehicles.

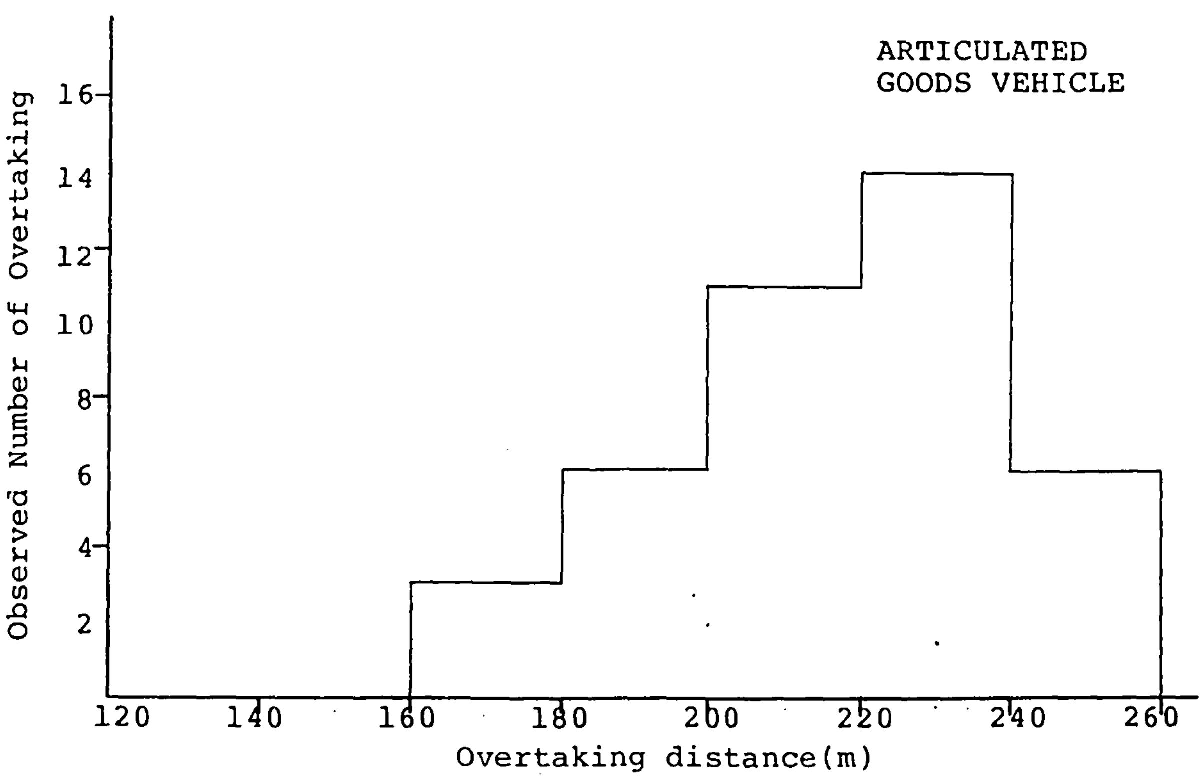

Figure 4.12.2-7 Distances of articulated vehicles overtaking slow moving vehicles.

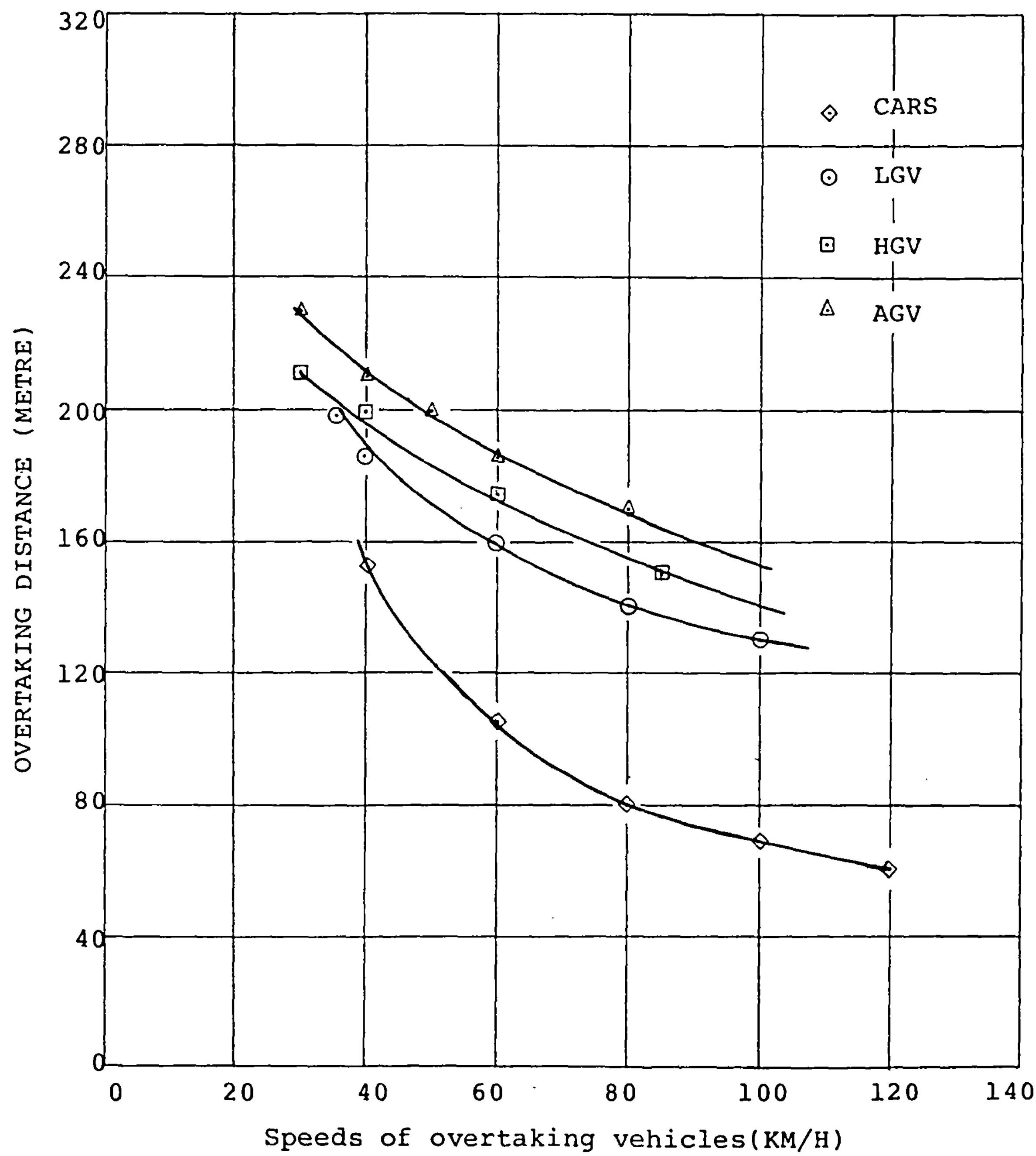

Figure 4.11.2-8 Overtaking and passing distances for different vehicle types.

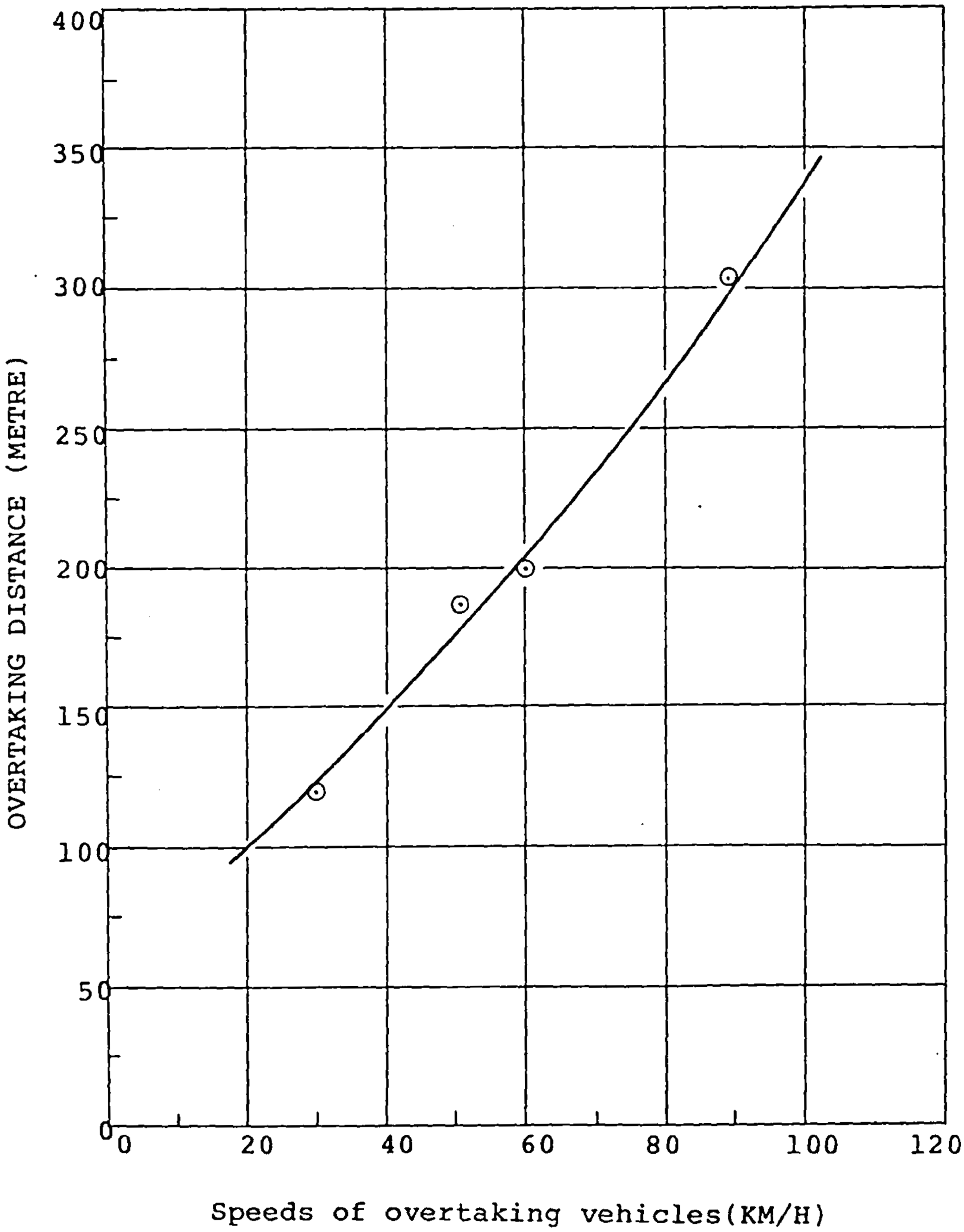

Figure 4.11.2-9 Overtaking and passing distances by cars at speed of overtaken vehicle.

taking is seconds. A heavy goods vehicle travelling at an average speed of 90 km/h requires an average distance of 145m to overtake. Therefore, the time taken by a heavy goods vehicle to overtake is 5.80 seconds which means that its lane occupancy is 5.80 seconds. This indicates that a passenger car will experience a delay of 2.80 seconds when deciding to overtake while the overtaking lane is occupied by a heavy goods vehicle. Similarly an articulated goods vehicle will require 6.40 seconds to overtake while travelling at 90 km/h, which indicates that a passenger car driver will experience a longer delay of 3.40 secs. before overtaking when an articulated goods vehicle is occupying the overtaking lane.

On two-lane motorways a higher number of heavy and articulated goods vehicles occupy the far side lane to overtake other vehicles travelling on the near side lane. This results in retarded maintenance of the desired speed of smaller and faster vehicles and increases the total time of a journey. At higher percentages of heavy and articulated goods vehicles in the total flow, an increase of overtaking lanes occupancy by heavy goods vehicles was observed. This affects average speeds and consequently reduces the total flow that can be accommodated by each lane per hour. This means a reduction in the total capacity per lane and a lower service level as a result of the presence of heavy and articulated goods vehicles.

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A comparison between different types of vehicles is given in figures (4.12.2-10) to (4.12.2-18), which shows the time required by individual vehicle types to overtake at the same average speed of travel. At lower speeds, the difference in time required to overtake between passenger cars and articulated goods vehicle becomes significant, which reflects the effect of heavier vehicles on traffic flow performance.

The recent increase in longer and wider heavy goods vehicles has led to serious operational problems. The limited ability of these large vehicles to maintain speed especially on long grades causes following motorists to initiate passing manoeuvres, often in hazardous situations. Also, large vehicles frequently restrict overtaking opportunities and make these manoeuvres difficult and hazardous. These operational problems manifest themselves in reduced levels of service, delay and increased overtaking attempts, as well as in aborted passes and greater driver frustration. These delays are small and have only a marginal effect on average speed at low percentages of heavy commercial vehicles, but the delay effects increase with the increase in heavy commercial vehicle percentage. Average delays increase dramatically where passing and overtaking is restricted by large numbers of slower moving vehicles occupying the far side lane.

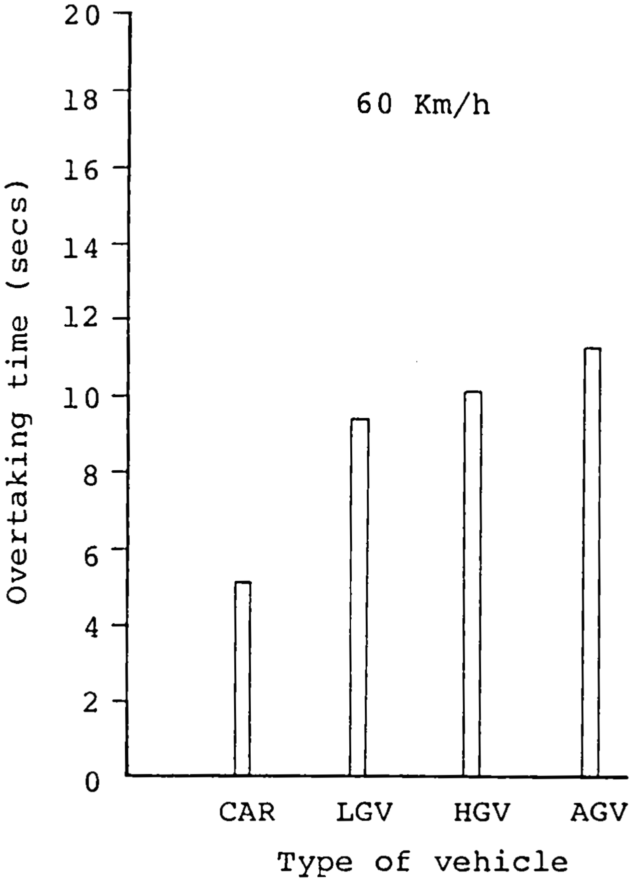


Figure 4.11.2-10

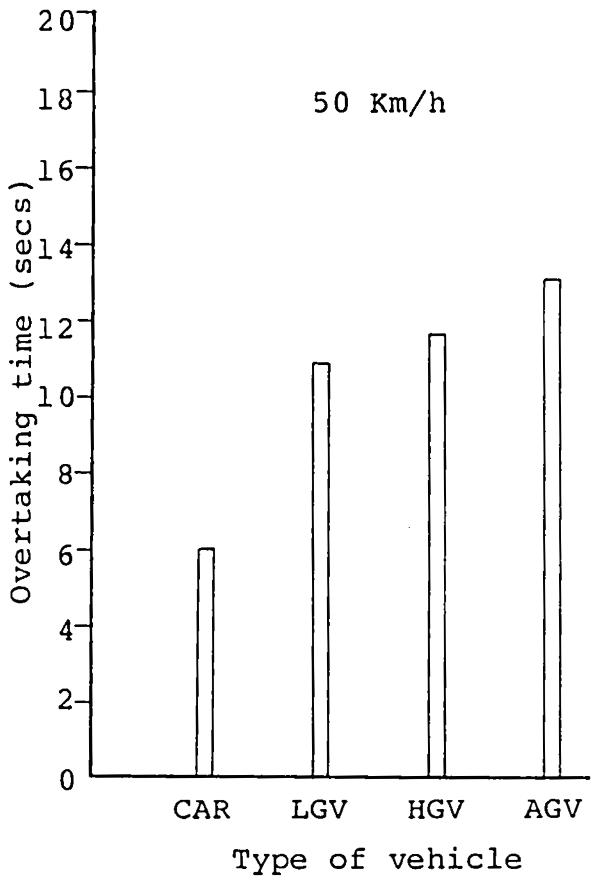


Figure 4.11.2-11

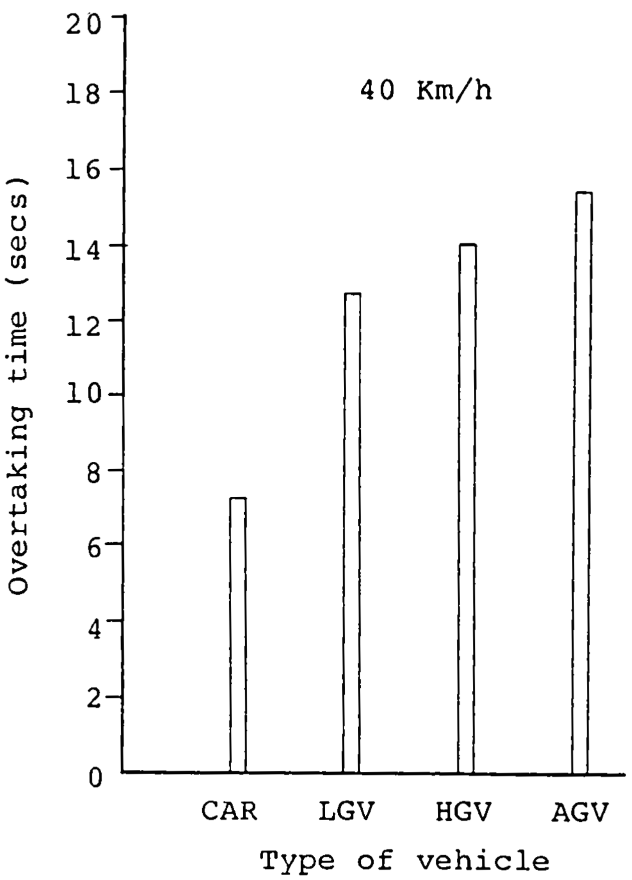


Figure 4.11.2-12

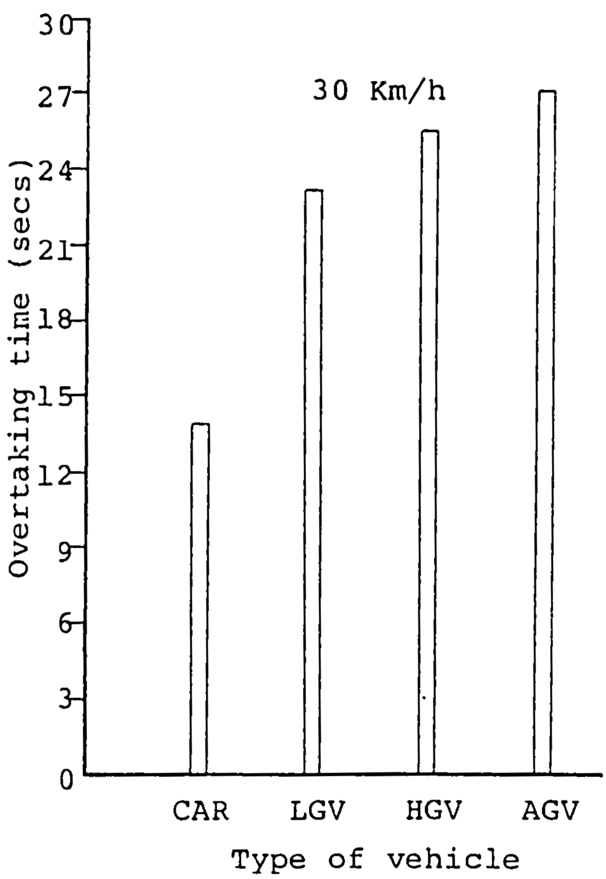


Figure 4.11.2-13

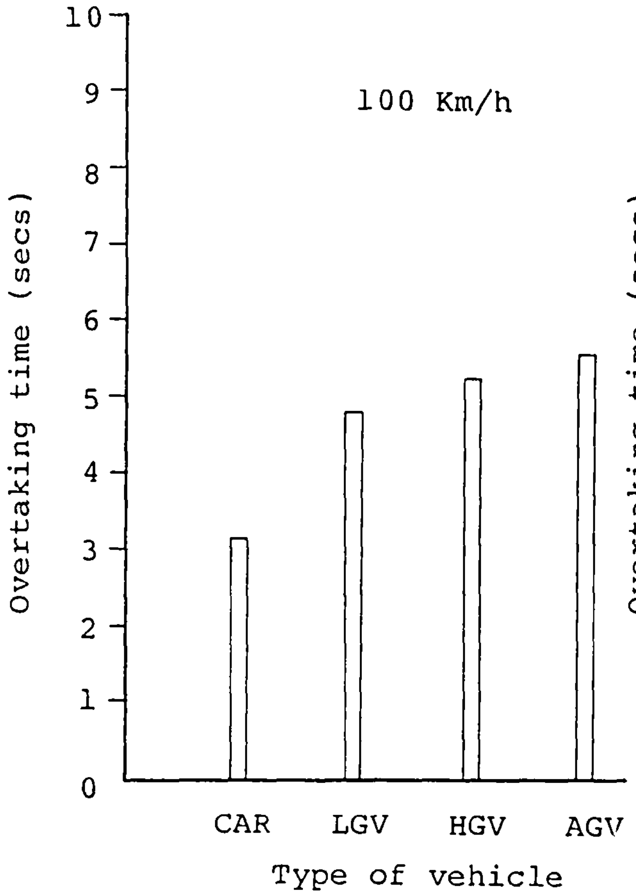


Figure 4.11.2-14

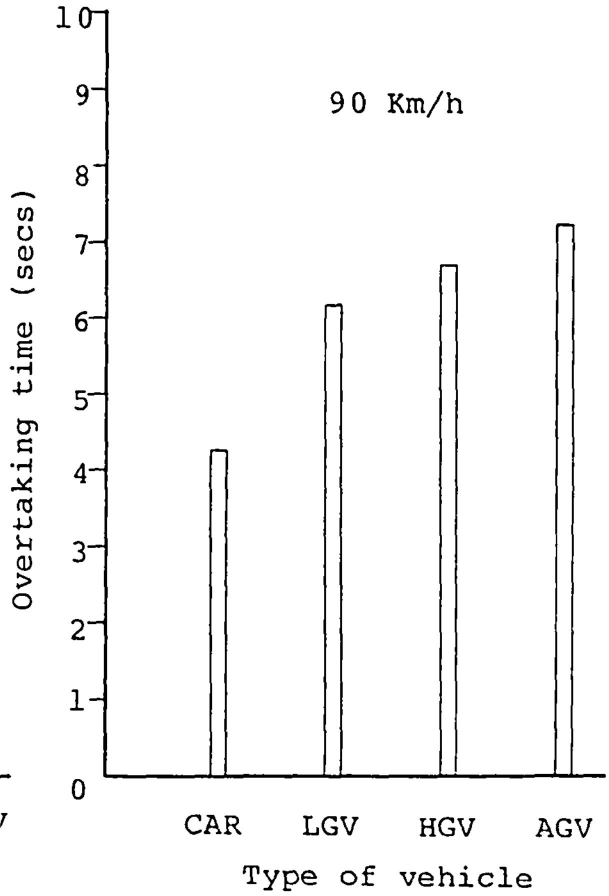


Figure 4.11.2-15

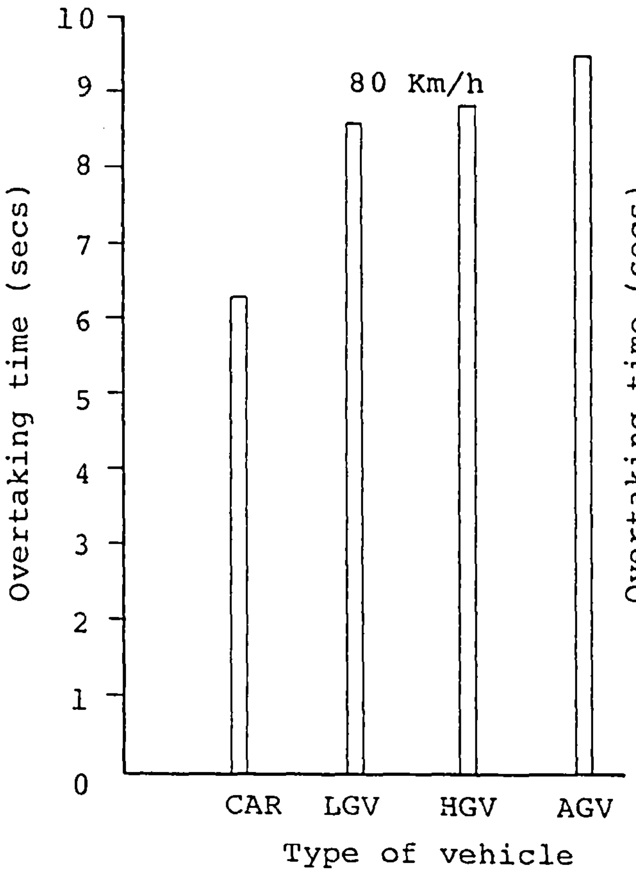


Figure 4.11.2-16

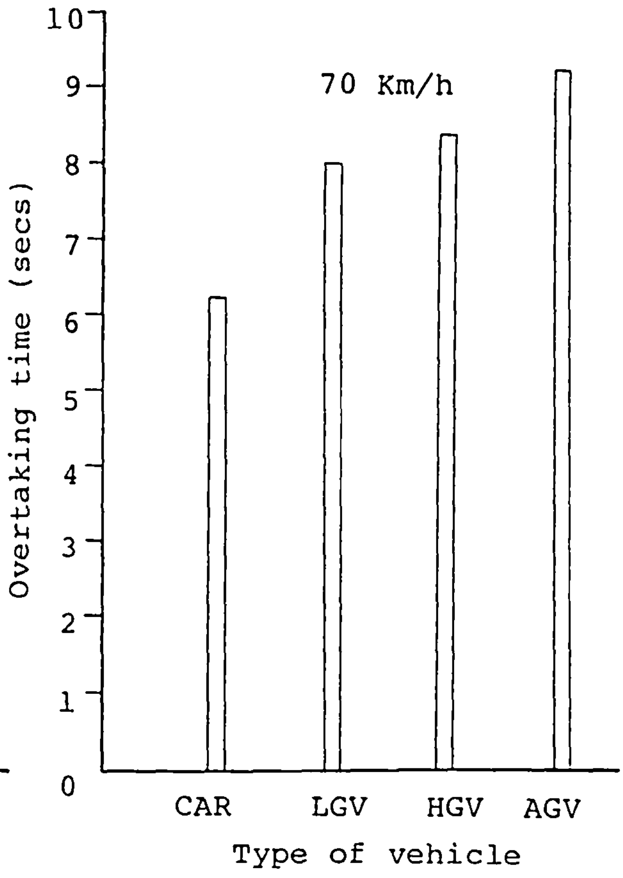


Figure 4.11.2-17

Overtaking criteria for a two-lane motorway showed that the average time required for passenger cars travelling at an average speed of 90 km/h (56 m/h) to overtake other types of vehicles travelling at an average speed of 40 km/h (25 m/h) is 9 seconds, while the average time required for light, heavy and articulated goods vehicles to overtake other vehicles travelling at 40 km/h (25 m/h) is 17.8 seconds. Also the time and distance required to overtake increase as the overtaken vehicle's length and speed increase.

The variations in time and distance required for vehicles to overtake are due to their sizes and manoeuvring performances (accelerating and decelerating capabilities).

Speed flow relationship results showed the effect of the percentages of heavy vehicles in the total flow on speed and lane capacity. Lane capacity is reduced as the percentages of heavy commercial vehicles increase. Also higher speeds are maintained when low percentages of heavy commercial vehicles are present in the traffic flow. This clearly indicates that at lower percentages of heavy commercial vehicles in the traffic stream, higher values of speeds can be maintained and consequently higher traffic values can be accommodated per lane/hour which will result in the increase in the capacity of the roadway.

5

Effects of Vehicle Type and Traffic Growth on Traffic Performance in Saudi Arabia

CHAPTER FIVE

Effects of vehicle type and traffic growth on traffic performance in Saudi Arabia

5.1 - Introduction

In the early seventies oil revenue began to grow and Saudi Arabia became one of the richest states in the Middle East. Therefore, the government initiated the implementation of ambitious five-year development plans in which the objective was to change the country from a pre-industrial society to a modern industrialized country. This rapid development exerts pressure on all public utilities and facilities including the transportation system, especially in urban areas. As one component of the development plans, the government is trying to improve the transportation system in the country as a whole, and in major cities such as Jeddah, Riyadh, Dammam, Makka and Al-Medina.

As a result of this sudden expansion in the country's development plans, many difficulties have had to be overcome. It has been necessary to design and construct major highways between important cities and industrial areas, on the one hand, and to try to solve urban traffic problems on the other.

Petroleum is to Saudi Arabia much of what the automobile industry has been to the United States and Europe. As the automobile is a leading segment of the North American and European economy and as its products and finances have shaped their society, so has petroleum extraction played a similar although larger role in Saudi Arabia.

Oil, like the car, is the instrument of change, but it is not the change itself. As cars wrought such new social development as the dispersion of cities, the growth of suburbs and the creation of an immense road system, oil has wrenched most of Saudi Arabia out of a nomadic or village life. Now it is financing the metamorphosis of the country into a twenty-first century state, if not necessarily in the western style (119).

Construction has occurred in every city, making familiar neighbourhoods unrecognizable in six months. Entire new cities were built in 1977, on what historically were oceans of vacant sand, such as the city of Al-Jubail on the Arabian Gulf in the Eastern Province and the city of Yanbu on the Red Sea in the Western Province, while older urban centres are becoming high-rise cities, expanding skyward almost as rapidly as outward. The scale of building and expansion in road networks throughout Saudi Arabia has been so great that the giant construction crane is only half-humorously referred to as the national bird of the Kingdom (119).

This surge of construction has produced traffic congested central city areas fed by heavily congested, frequently paralysed arterial roads which are surrounded by sprawling, half-empty suburbs. The momentum of urban growth propels cities outward, with new development spreading rapidly to established boundaries, rolling over them and creating new but ephemeral outer limits.

Saudi cities are growing not merely in physical proportions but in human dimensions also. The Saudi citizen is moving to these burgeoning urban centres in large numbers and he is joined by a large foreign labour force estimated at about two and a half million people from all over the world.

In keeping pace with the progress achieved in other sectors in the Kingdom, the transportation and communications sector has witnessed amazing changes during the last few years. As the role of transportation and communications has continued to expand, it has played a key role in the over-all development of the United Kingdom's economy and has had a significant impact on the lives of the citizens. However, traffic laws and standards have received insufficient attention. Until recently very few traffic regulations and standards were published or implemented by different local authorities. Two main reasons can be given to explain the size of the traffic problem, though other reasons will be discussed later:

i) The absence of a traffic department leading to traffic rules being changed by different government authorities.

ii) The lack of scientific research in the field of traffic studies, where traffic design standards are left to foreign consultants.

5.2 - Development of road network

5.2.1 - Introduction

Because of the dramatic increase in traffic throughout the Kingdom, the Ministry of Communications and Transportation has undertaken a comprehensive highway development programme. In recent years this development programme has focused increasingly not only on the quality of the road network but also on the quality of the system. The present programme emphasizes the following objectives:

(1) Expanding the road network to reach all the principal areas in the Kingdom and to provide access to as many towns and villages as possible;

(2) Widening existing roads and constructing more dual carriageways and expressways in areas with traffic congestion;

(3) Improving preventive maintenance and road-safety measures.

Saudi Arabia is a large and sparsely populated country which covers over two and a half million square kilometres and a well-planned road system is difficult to achieve, though vital to the over-all development of the country.

In addition to the vastness of the Kingdom, Saudi Arabia is characterized by extremes of climate and geography, ranging from high rugged mountains to rocky plains and to vast deserts of sand. In carrying out development plans it has been necessary to deal with the complex problems caused by such extremes. To best meet the requirements dictated by the country's physical environment, the most modern international specifications have been adopted for highway design and construction. They were published by the Ministry of Communications and Transportation in a guide-line manual with the general requirements as well as detailed specifications for the use of consultants and contractors engaged in designing and building the road network.

The country is divided into five regions in order to assist such development programmes, including road network and transportation facilities. Main roads were planned to connect the various sections of each region and then the various regions with each other, and were built with assistance from both Saudi and foreign consultants. These regions are:

a - Eastern Province

b - Western Province

c - Northern Province

d - Middle Province

e - Southern Province

Roads projects have played a major role in the Kingdom's Five-Year Development Plans. The development of the road network and other transportation elements are vital to the successful economic development of the Kingdom. During the first plan 1390-1395 H (1970-1975 A.D.), most of the districts and main cities were linked by a network of paved roads. By the end of this plan, 1219 km. of asphalt roads and 8510 km. of agricultural roads had been built.

With the second development plan 1395-1400 H, (1975-1980 A.D.), the major projects had been completed. The total length of paved roads had grown to 21,583 km. and agricultural roads had increased to 24,186 km. Most of the Kingdom's districts were now connected by a modern road network.

By the end of the third development plan in 1405 H (1985 A.D.), the Kingdom had 29,655 km. of paved highways. Agricultural roads had been increased to 50,655 km.

The fourth development plan has set a target for increasing the road network in the Kingdom to 116,000 km. which will include 35,000 km. of paved highways (primary, secondary and feeder roads) and 81,000 km. of agricultural roads.

5.2.2 - Urban roads

As a result of the rapid social and economic growth and development which has taken place during recent years, the Kingdom's cities have expanded at an equally rapid rate. Many cities have grown so rapidly that in just a few years they have expanded to many times their original size.

Traffic has grown just as rapidly and has exceeded all expectations, leading to traffic congestion on major city streets throughout the Kingdom.

Major inter-city road development and expansion has taken place. The network of inter-city paved roads expanded at an average rate of 10.1 per cent yearly from 8,000 km. in 1970 to 30,000 km. in 1985 (121). Figure (5.2.2-1) shows the development and expansion of the urban road network. Nearly all towns and cities in the Kingdom are now served by dual-carriageway roads, many of which contain three lanes and four lanes in each direction with a two-lane additional service road in each direction.

Major intersections were widened and developed to accommodate higher traffic volumes. Different junction designs have been adopted and implemented to solve the problem of urban junction congestion. Grade-separated

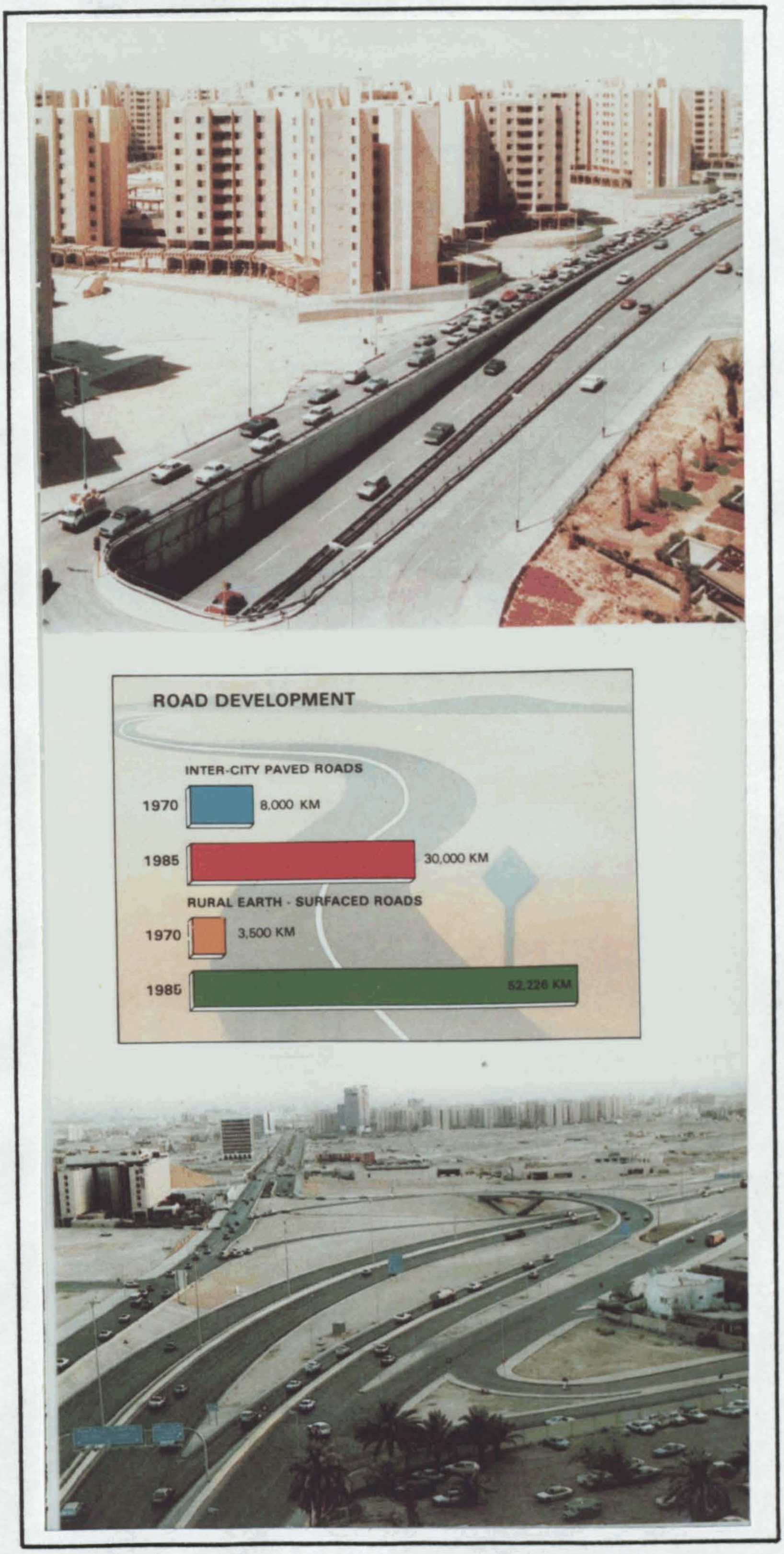


Figure 5.2.2-1 Urban Road Development and Expansion.

(From reference No. 120)

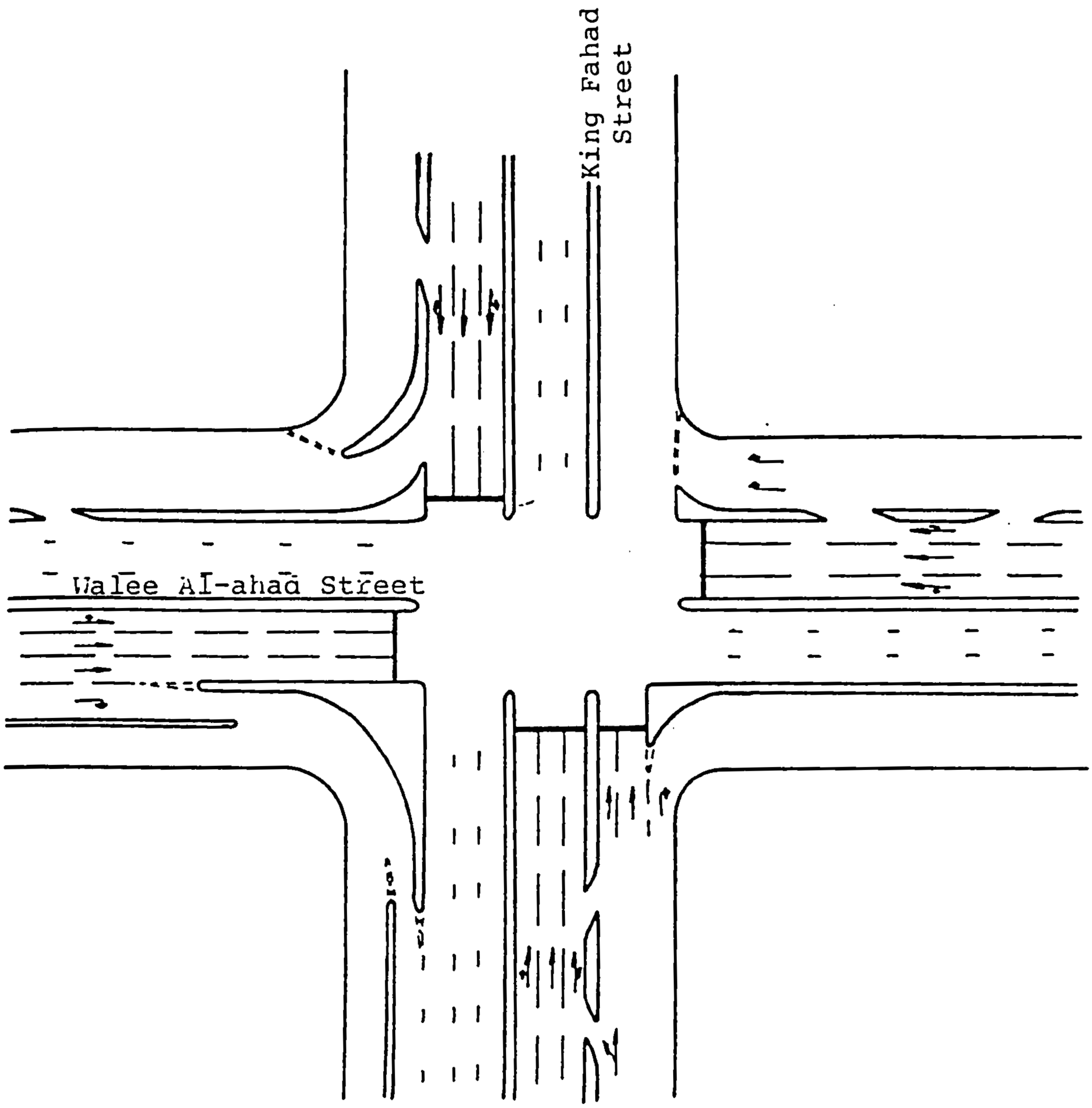


Figure 5.2.2-2 Traffic Signal Approach with filtering exit (Jeddah).

(Reproduced from reference No.122)

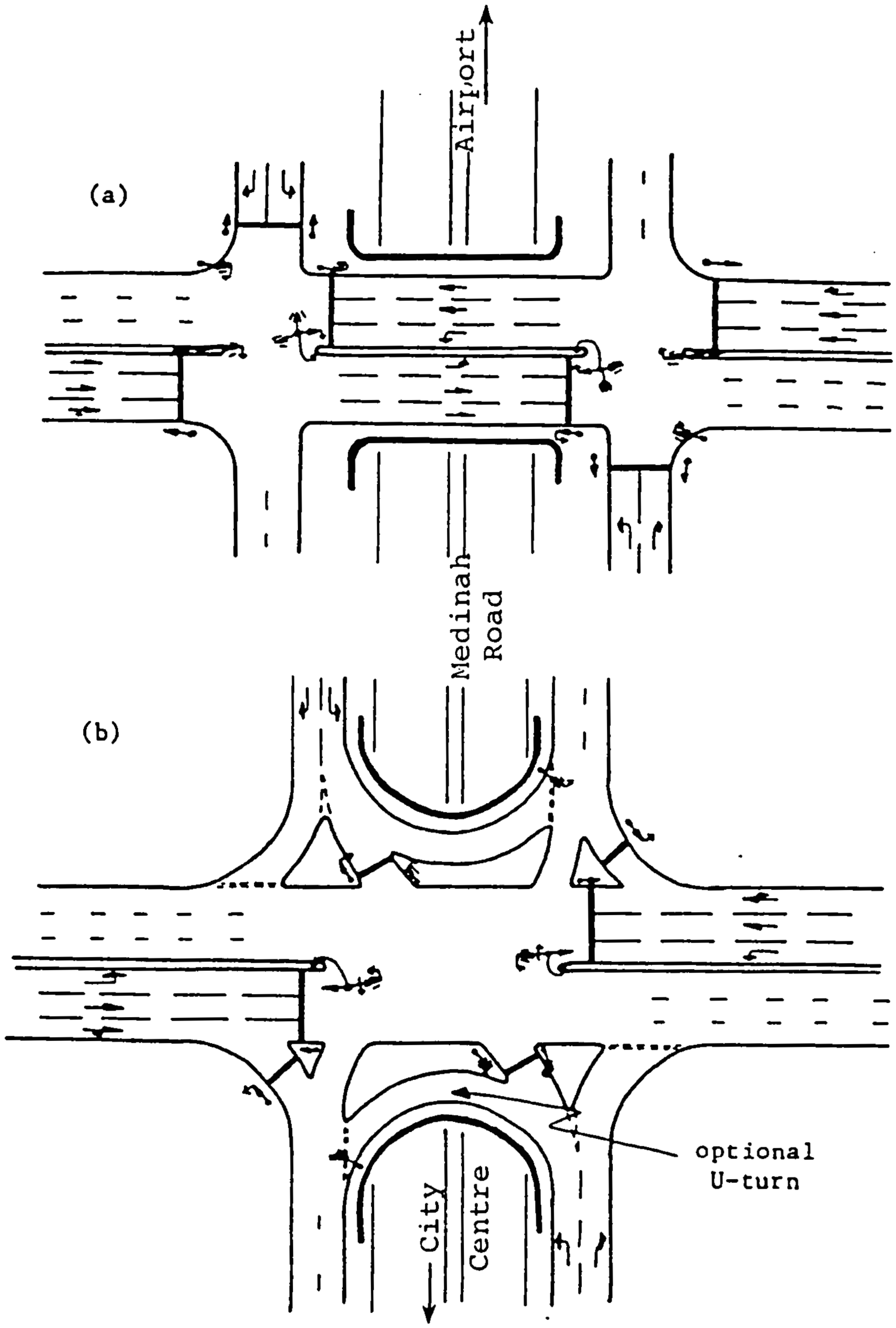


Figure 5.2.2-3 Layouts of Signalized Diamond Intersection (Medinah-Airport Road) Jeddah.

(Reproduced from reference No.122)

junctions, many of which are "diamond" interchanges with signal control in some cases, is one of the solutions; other major arterials have been totally converted to elevated urban roads with three lanes in each direction ex-extending to 17 km. in some places. Under-passes and free-flow unsignalized slip roads are another example adopted to increase junction capacity. Figures 5.2.2-2 and 5.2.2-3 show examples of junction layout design from the city of Jeddah.

The design of urban intersections took into account traffic composition in Saudi Arabia. As mentioned earlier, multi-million Saudi-Riyals projects were constructed in and around the cities,which require the use of heavy commercial vehicles. Heavy commercial vehicles had no standards or regulations in the early stages of the construction boom in the Kingdom. Also the presence of inexperienced drivers was another factor which increased congestion and accidents in different locations of urban network. For example, in the Eastern Province where major oil reservoirs are located, oil companies use longer and heavier types of commercial vehicles to transfer their equipment, in many cases through the cities. To add to the problem of traffic composition in urban networks, the Saudi Public Transport Company (SAPTCO) was established in Spring 1979 and 600 buses (43 passenger capacity) were introduced in every major city.

This is in addition to the unregistered buses of small private companies. Most large companies, particularly foreign-based construction or service firms, use some form of group transportation for their employees and their families.

These above-mentioned factors led to a dramatic increase in urban road traffic activity which exerted high pressure on the road network. To alleviate these problems and facilitate the movement of traffic in the cities, ring roads have been constructed around several major cities.

The ring roads have been built to expressway standards and include service roads in addition to three-lanes in each direction, interchanges, bridges and underpasses with accompanying lighting, landscaping and safety provisions.

The modern ring road concept envisages easy access to outlying areas without any hindrance from traffic lights and the other barriers or inter-urban city streets. These ring roads helped in diverting through-traffic, especially heavy commercial vehicles, away from more congested city centres. Also ring roads facilitate more rapid movement between various sections of the city which is particularly important for public service vehicles such as fire-fighting equipment, ambulances and police.

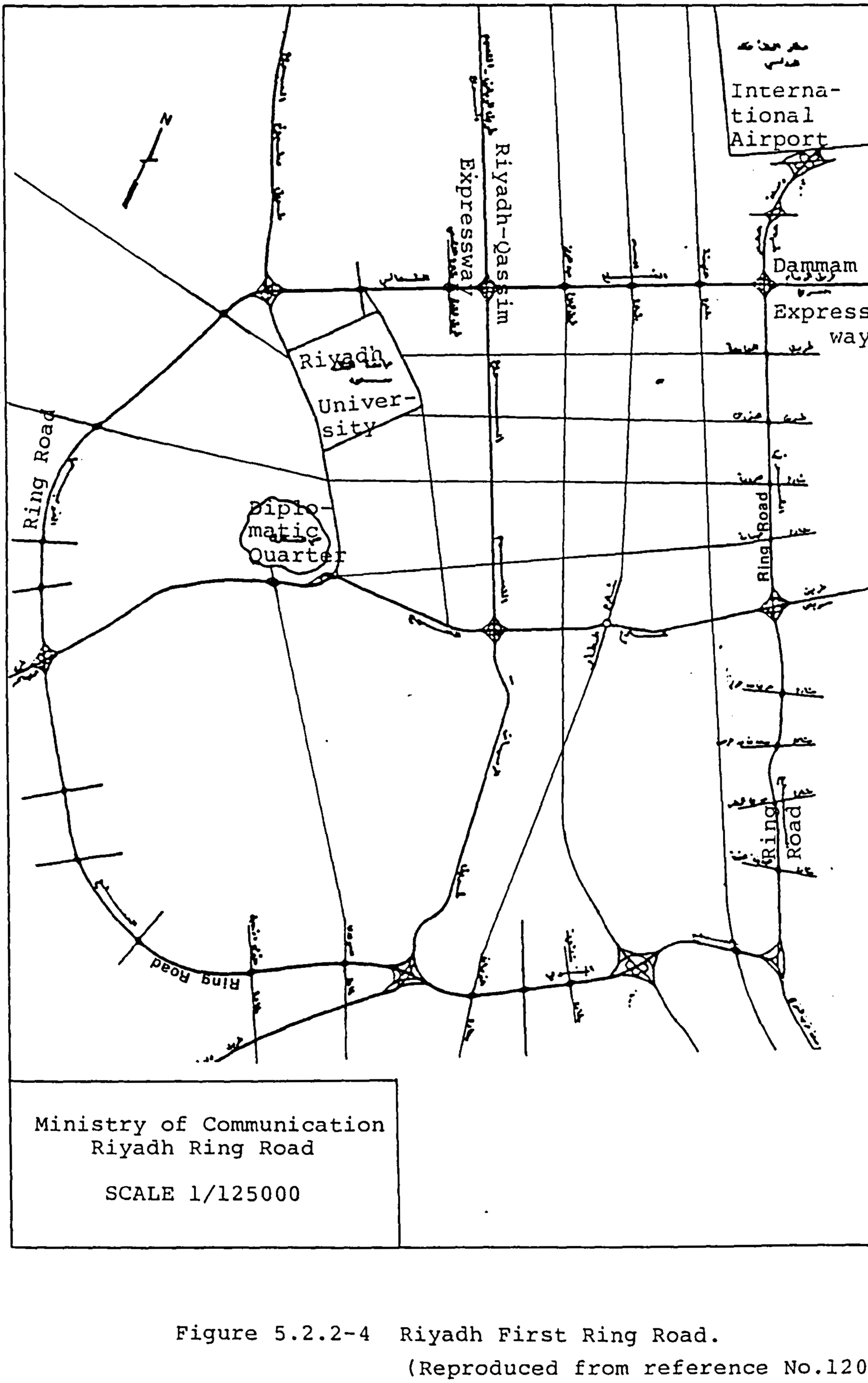


Figure 5.2.2-4 Riyadh First Ring Road.
(Reproduced from reference No.120)

As an example, Figure (5.2.2-4) shows Riyadh's first ring road which passes through the urban areas of the city. The total length of this road is 93 km. It consists of three lanes in each direction and an 8m wide central reservation making the total width of the road 100m. This ring road has 42 intersections with other major routes. The problems which would arise from such intersections has been solved by the use of different designs, such as diamond interchanges, bridges or underpasses (see Figure 5.2.2-4). The ring road is also provided with lighting fixtures, landscaping and all other services.

5.2.3 - Rural roads

The rural road network expanded from 3,500 km. in 1970 to 52,226 km. in 1985, at an average annual growth of 20.5 per cent. These roads serve more than 8,000 cities, towns and villages. Early rural road designs were a single carriageway (two-way) undivided. Then the Ministry of Communications carried out an extensive programme to develop and modernize rural road networks in the Kingdom. The road programme which the Ministry of Communications have approved consists of major features as follows;

1 - The reconstruction and widening of old roads;
2 - The implementation of high road design standards;
3 - The planning and execution of road projects in collaboration with experienced foreign institutions.

The primary road programme objectives have conformed to the content of the Kingdom's five-year development plans which the government implements. These objectives are as follows:

a. Promoting the growth of the various sectors such as increasing agricultural and industrial production; elevating the standard of health, educational and social services, reducing costs of transport and communications required for economic and social activities by the co-ordination between road development programme and public services development programme in order to realize a balanced investment ensuring maximum possible national income in all sectors;

b. Strengthening and supporting the national integrity and regional economic growth, by linking all villages whose population exceed 10,000 people with primary roads either directly or by road connections;

c. Meeting the increasing growth in traffic with the lowest possible costs, by creating an equilibrium between road maintenance and vehicle fuel-consumption and considering the time element in the cost of travel.

At the end of the first development plan (1970-1975), most of the main regions and cities had been connected by primary and secondary asphalt roads of which the total length had reached 12,169 km. The total length of agricultural feeder roads was 8,077 km.

At the end of the second development plan (1975-1980), 21,583 km. of primary and secondary asphalt road projects had been completed and 24,186 km. of agricultural feeder roads.

The third development plan (1980-1985) included an increase in the total length of roads to 6,000 km. (excluding agricultural feeder roads), together with the various improvements made to the existing projects, including the expansion of some roads, the construction of bridges and mountain passageways and the conversion of other roads into expressways to allow for the increase in traffic. It was anticipated that this goal might be exceeded by the end of 1985.

As a result of such achievements it was possible to execute the third stage,which was to develop the services and safety requirements on long distance roads, such as using guides, indicative and information signs and markings, and encouraging the process of placing advertisement hoardings on both sides of the road. The third stage aimed to further increase green areas around the roads by the planting of trees and desert plants which do not require continuous watering. It was hoped that these topographical scenes would stimulate drivers to remain alert over long distances and to withstand the hot climate. Also the third stage included the construction of rest areas with complete facilities for car parking and servicing, restaurants and sleeping units (motels).

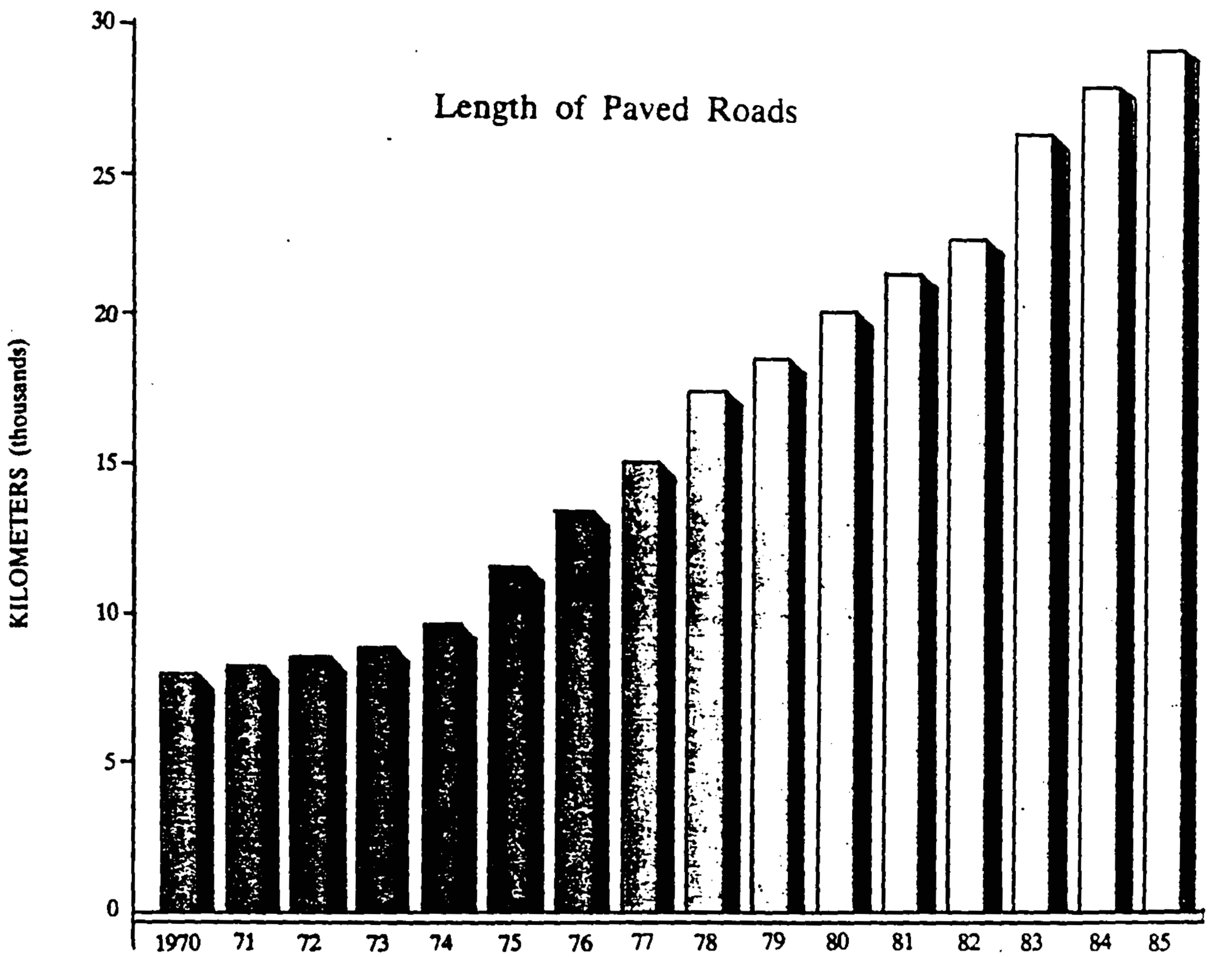


Figure 5.2.3-1 Length of rural paved roads constructed between 1970-1985, Saudi Arabia.

(Reproduced from reference No.120)

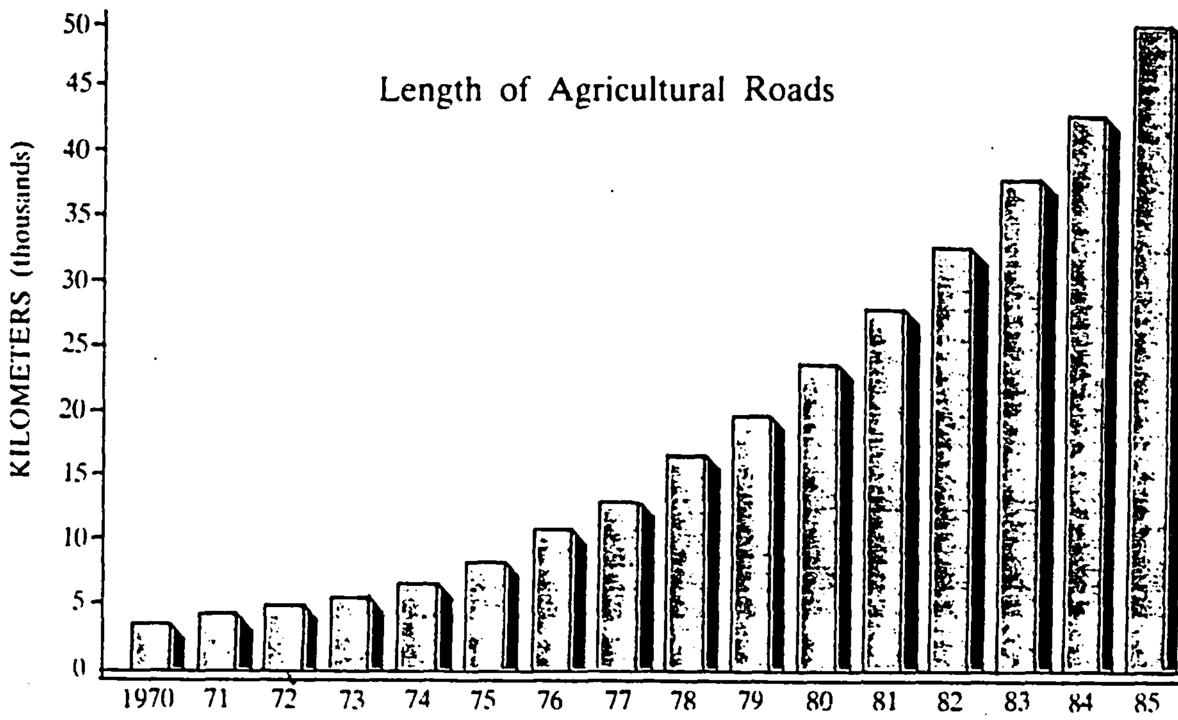


Figure 5.2.3-2 Length of Agricultural Roads constructed between 1970-1985, Saudi Arabia.
(Reproduced from reference No.120)

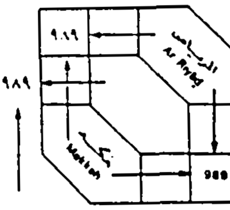
جدول المسافات الكيلومترية بين المدن الرئيسية بالمملكة

TABLE OF DISTANCES IN KILOMETRES BETWEEN IMPORTANT SAUDI ARABIAN CITIES

a	via Jiddah road	١ عن طريق جدة
b	Al Madinah road	٢ عن طريق المدينة
c	Ar Riyāḍ road	٣ عن طريق الرياض
d	Ar Riyāḍ–At Ṭā'if road	٤ عن طريق الرياض الطائف (الحجاز)
e	Dhahran road	٥ عن طريق الظهران
f	Buraydah road	٦ عن طريق بريدة

المسافات المبينة على هذا الجدول هي على الطرق المسفلتة والطرق الجاري تنفيذها . وقد أخذت من حدود كل مدينة .

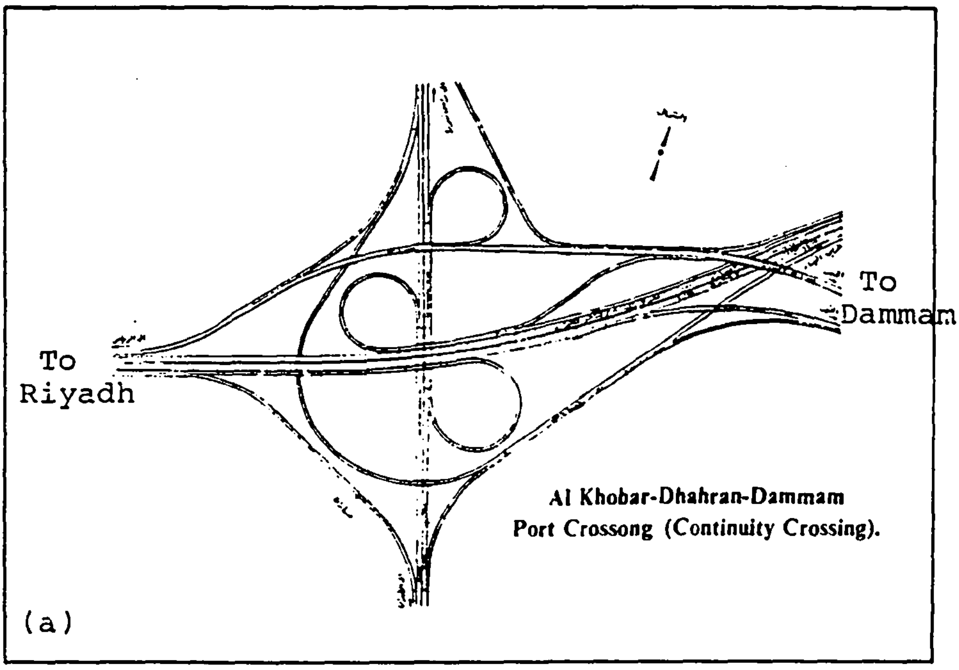
These distances refer both to paved roads and roads under construction. Distances are measured between city boundaries.



	Abhā	Ad Dammām	Al Hufūf	Al Kharj	Al Kuwayt	Al Madīnah	Ar Riyāḍ	As Sulayyil	Aṭ Ṭā'if	Buraydah	Dhahran	Ḥā'il	Jiddah	Jīzān	Makkah	Najrān	Sakākah	Tabūk
Ad Dammām	d 1811																	
Al Hufūf	d 1772	159																
Al Kharj	d 1628	c 551	c 412															
Al Kuwayt	e 2346	441	594	988														
Al Madīnah	e 1078	d 1903 / f 1459	d 1764 / f 1320	d 1520 / f 1076	f 1894													
Ar Riyāḍ	d 1444	467	328	84	902	d 1436 / f 992												
As Sulayyil	c 2000	c 1023	c 884	480	1458	d 1992 / f 1548	558											
Aṭ Ṭā'if	543	1368	1229	c 985	1803	535	901	c 1457										
Buraydah	d 1370	c 953	c 814	c 570	1388	541	488	c 1042	d 827									
Dhahran	d 1897	14	146	c 537	449	d 1889 / f 1446	453	c 1009	1354	c 939								
Ḥā'il	d 1876	1215	1073	875	f 1693	398	746	f 1347	982	284	1201							
Jiddah	703	1528	1388	c 1145	1963	424	1061	1617	160	d 987 / c 985	1514	822						
Jīzān	188	d 2089	d 1960	d 1716	c 2534	e 1266	d 1632	c 2188	731	d 1558	d 2085	d 1863	891					
Makkah	631	1466	1317	c 1073	1891	447	989	c 1545	88	d 915	1442	894	72	819				
Najrān	247	1727	1594	d 1775	c 2593	1346	1266	c 2247	810	1736	1713	d 1922	970	435	898			
Sakākah	e 1828	315	156	568	750	f 1476	484	1040	1385	e 970	301	f 1276	1546	c 2116	1473	c 2176		
Tabūk	b 1784	d 2589 / f 2121	d 2460 / f 1982	d 2206 / f 1738	f 2556	688	d 2122 / f 1654	d 2678 / f 2210	1221	b 1203	d 2575 / f 2107	f 1506	1110	b 1952	1133	b 2011	f 2138	
Ṭurayf	c 3177	1272	e 1425	e 1817	1407	d 3169 / f 2725	1733	e 2289	c 2634	e 2219	1280	f 2524	c 2794	c 3366	c 2722	c 3424	e 1581	d 3858 / f 3387

Table 5.2.3-1 Distances in Kilometers between Major Cities in Saudi Arabia.

(Reproduced from reference No.121)



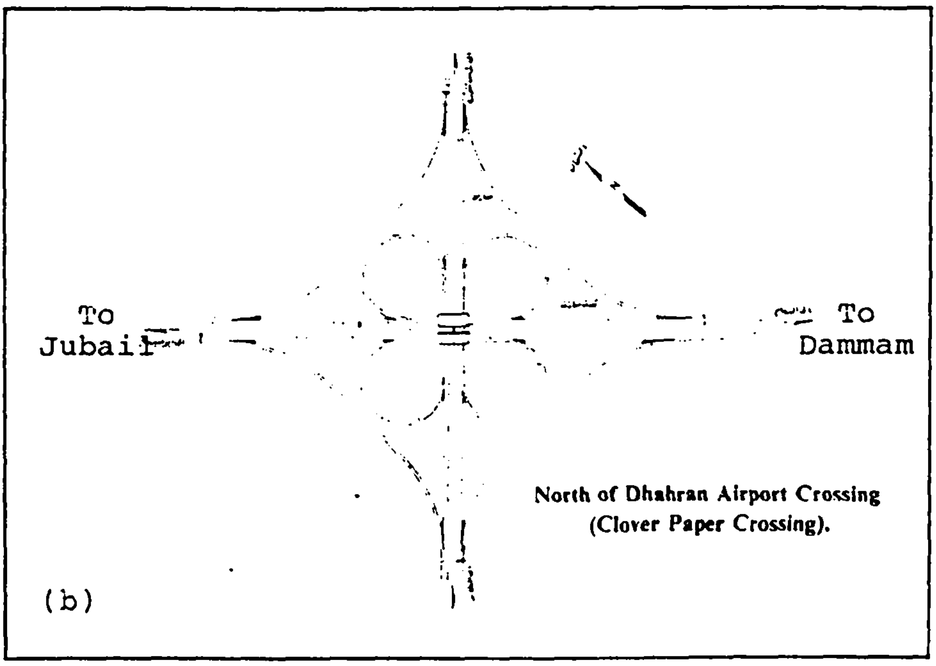
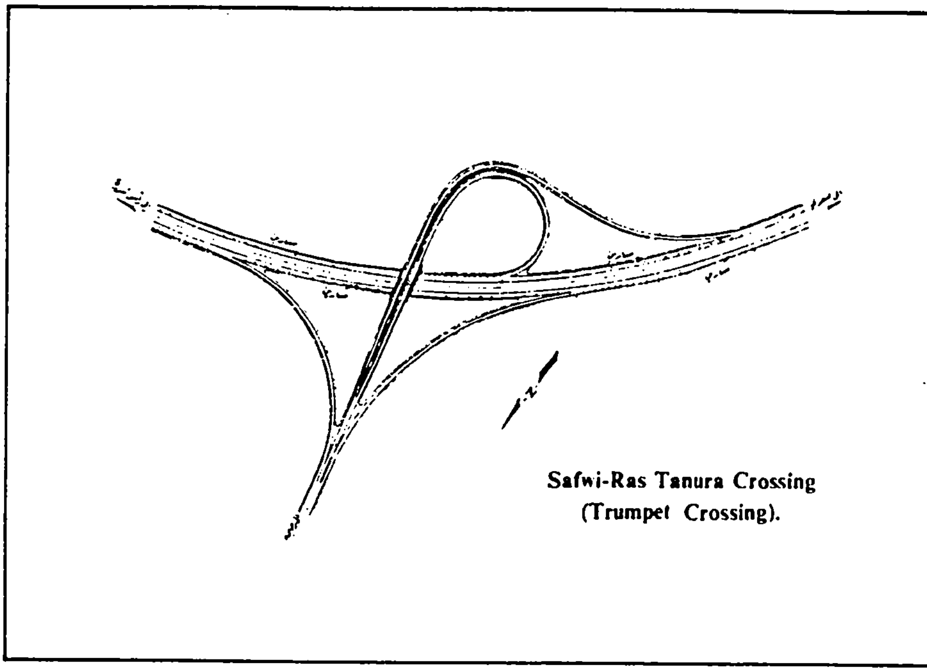


Figure 5.2.2-3 Expressway Diamond Interchange

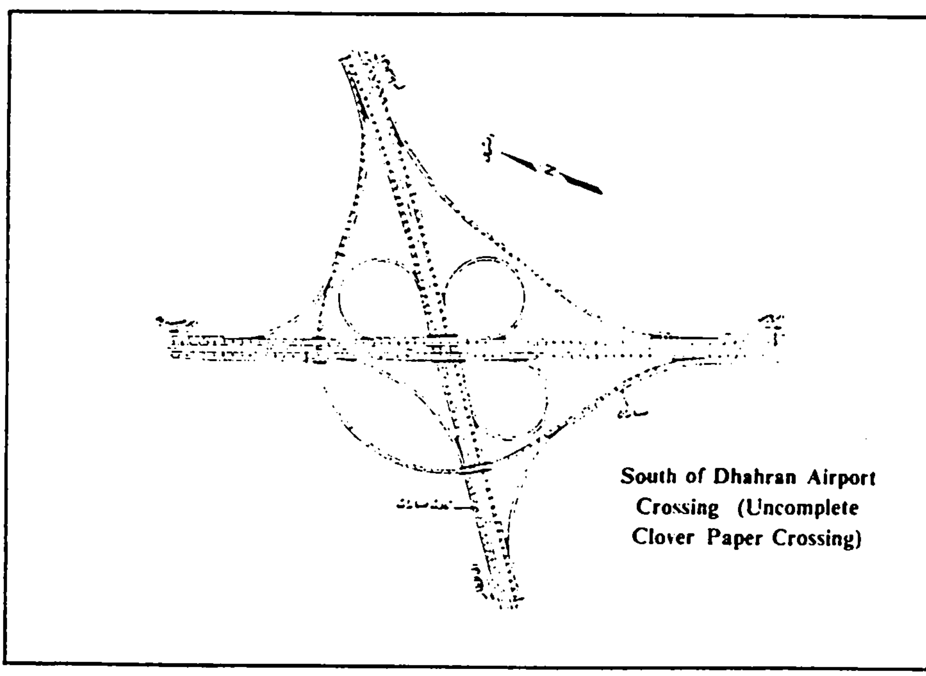
(a) Riyadh-Dammam Expressway

(b) Dammam-Jubail Expressway

(Reproduced from reference No.121)



(a)



(b)

Figure 5.2.2-4 Expressway Diamond Interchange

(a) Rass-Tanura-Safwi Expressway

(b) Dhahran Airport-Dammam Expressway

(Reproduced from reference No.121)

Figures (5.2.2-1) and (5.2.2-2) illustrates the increase in length of paced roads built from 1970 to 1985.

This stage also deals with widening and asphalting the shoulders of roads, organizing the process of overtaking, using painting and road signs and a huge maintenance programme. Furthermore, this stage includes a heavy commercial vehicle weighing programme to control loads, using fixed and mobile weigh bridges.

The fourth stage in the development plan has set a target of expanding the construction of road networks to 116,000 km. which will include 35,000 km. of paved highways (primary, secondary and feeder roads) and 81,000 km. of agricultural roads. The first expressway was completed at the end of 1979. It was the highway connecting the Holy City of Makka with Jeddah and had a total length of 75 km. This expressway consists of four-lanes in each direction with a 20m wide central reservation. The road carries a very high volume of traffic, especially in the periods of Hajj (Pilgrimage) when more more than two million people travel to Makka through Jeddah within a period of one to two months, mainly using the Makka - Jeddah expressway.

Then in 1983, the Makka - Al-Medina expressway was completed and opened to traffic. This road consists of three-lanes in each direction with a central reservation of 20 metres and a total length of 454 km. In 1984 the third expressway was opened connecting the capital, Riyadh, with Dammam on the East coast with a total length of 467 km. It has three lanes in each direction and a 20m wide central reservation. Then other expressways were opened during 1984 and 1985, and others are under construction and planned to open soon. The map in this section shows the rural network roads connecting cities and towns in the Kingdom. The map's key illustrates road categories, the completed road network and roads under construction. This map (1986) gives the latest information about rural roads in Saudi Arabia.

THE RAILROAD

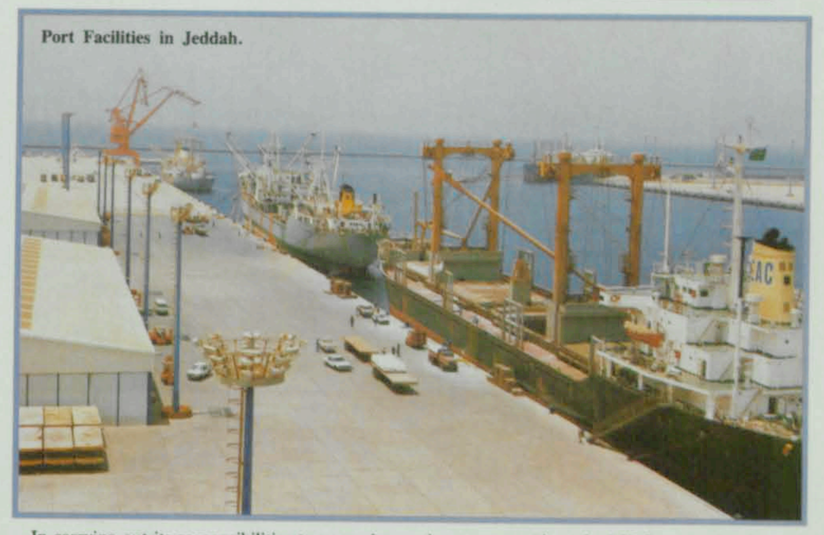
Saudi Railroad Transporting Freight Between Dammam and Riyadh.

In 1371 H. (1951 A.D.) the Riyadh-Dammam railroad began carrying both passengers and freight between the two cities. In 1403 H. (1983 A.D.) over 272,000 passengers and 1,698,000 tons of freight were carried on the line.

For the fiscal year 1404-1405 H. (1984-1985 A.D.) 666 million Saudi riyals (US$182.5) were allocated to the Saudi Railway Organization (SRO). The SRO is currently implementing a major expansion of its services by adding modern equipment. The railroad's capacity will be increased to 1,000 passengers per day. SRO is also currently studying the feasibility of constructing additional rail lines to connect other major centers within the Kingdom.

MARINE TRANSPORT

Port Facilities in Jeddah.



In carrying out its responsibilities to supervise marine transportation, the MOC works closely with both national and foreign shipping companies and participates in a variety of international maritime activities.

Facts and Figures

– The tonnage of the national commercial fleet has increased dramatically in recent years. In 1379 H. (1977 A.D.) the fleet carried 530,000 tons in 70 ships. By 1401 H. (1981 A.D.) this had increased to 3,250,000 carried in 166 ships. By 1404 H. (1984 A.D.) the totals were 5,597,000 tons carried by 372 ships. The total tonnage carried increased by over ten-fold in less than a decade.

– 147 licensed shipping companies are engaged in marine transportation in Saudi Arabia while 349 Saudi flag ships are now registered at Jeddah and Dammam ports.

PUBLIC TRANSPORTATION

A SAPTCO "Double-Decker" Bus.

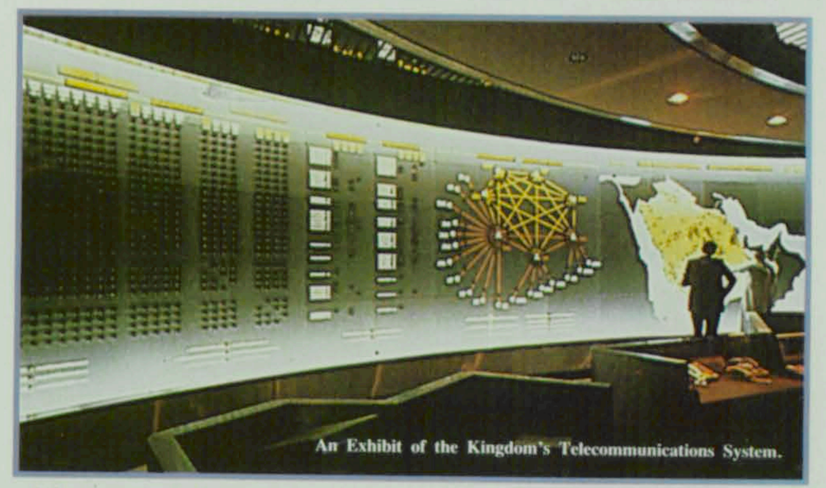
The Saudi Public Transport Company (SAPTCO) came into being in 1399 H. (1979 A.D.). With an initial funded capital of one billion Saudi Riyals (approximately US$300 million), the company began to provide bus service in the Kingdom's major cities. In addition, inter-city service was provided between the major metropolitan areas of the Kingdom.

During its first year of operations, SAPTCO carried 38.3 million intra-city passengers and 1.4 million inter-city passengers. By fiscal year 1404/05 H. (1984-85 A.D.) total ridership had risen to approximately 71.5 million. Since its inception the company has transported over 518 million passengers on intra-city services and 13 million on its inter-city routes.

SAPTCO has expanded its services regularly and now serves all the Kingdom's most populated areas. It has continued to upgrade and expand its bus fleet to meet changing demands. In addition to its regular services, SAPTCO plays a major role in the transport of pilgrims during the major pilgrimmage seasons.

The Ministry of Communications is responsible for the supervision of SAPTCO's operations. The Deputy Minister of Communications for Transport Affairs currently serves as the Chairman of SAPTCO's Board of Directors. SAPTCO has become a key agency in implementing the Ministry policy of improving the Kingdom's land transportation services.

POST, TELEPHONE AND TELEGRAPH



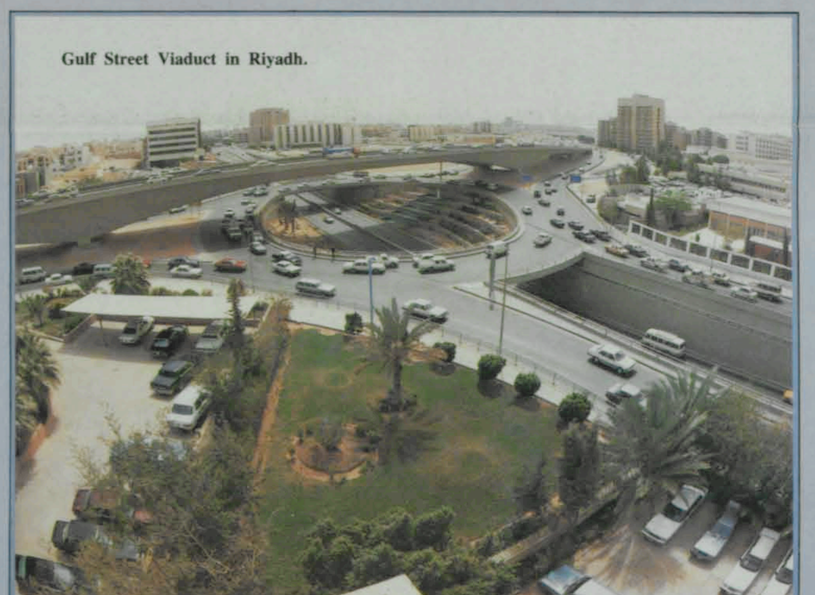
An Exhibit of the Kingdom's Telecommunications System.

A key component of any nation's communications infrastructure is its system to provide mail, telephone, and telegraph services. These services connect all parts of the Kingdom as well as providing vital links with the rest of the world. The system is administered by the Ministry of Post, Telephone, and Telegraph.

The Kingdom's accomplishments in this key field are summarized as follows:

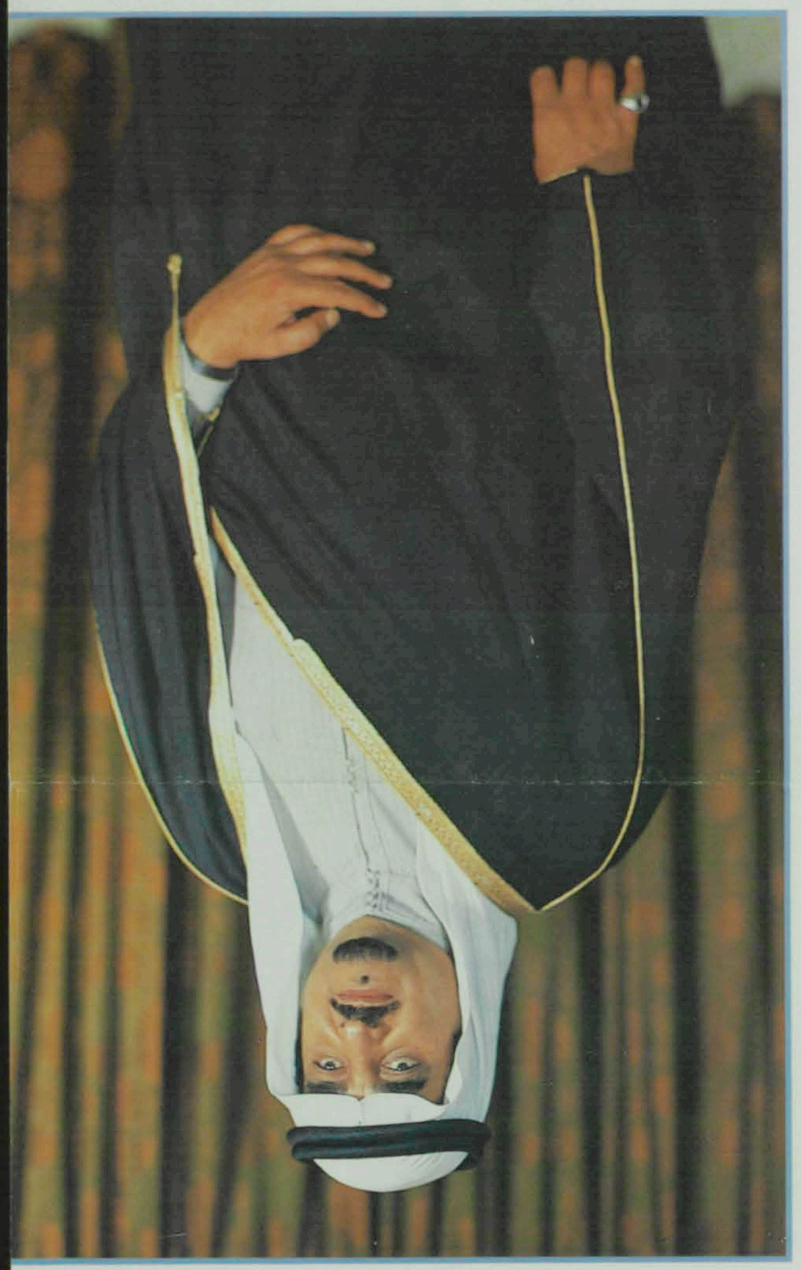
Number of Telephone Lines:	972,985
Number of Telex Lines:	16,543
Number of Automobile Telephones:	9,752
Number of Public Telephones:	4,646

TRANSPORTATIONS AND COMMUNICATIONS IN THE KINGDOM OF SAUDI ARABIA



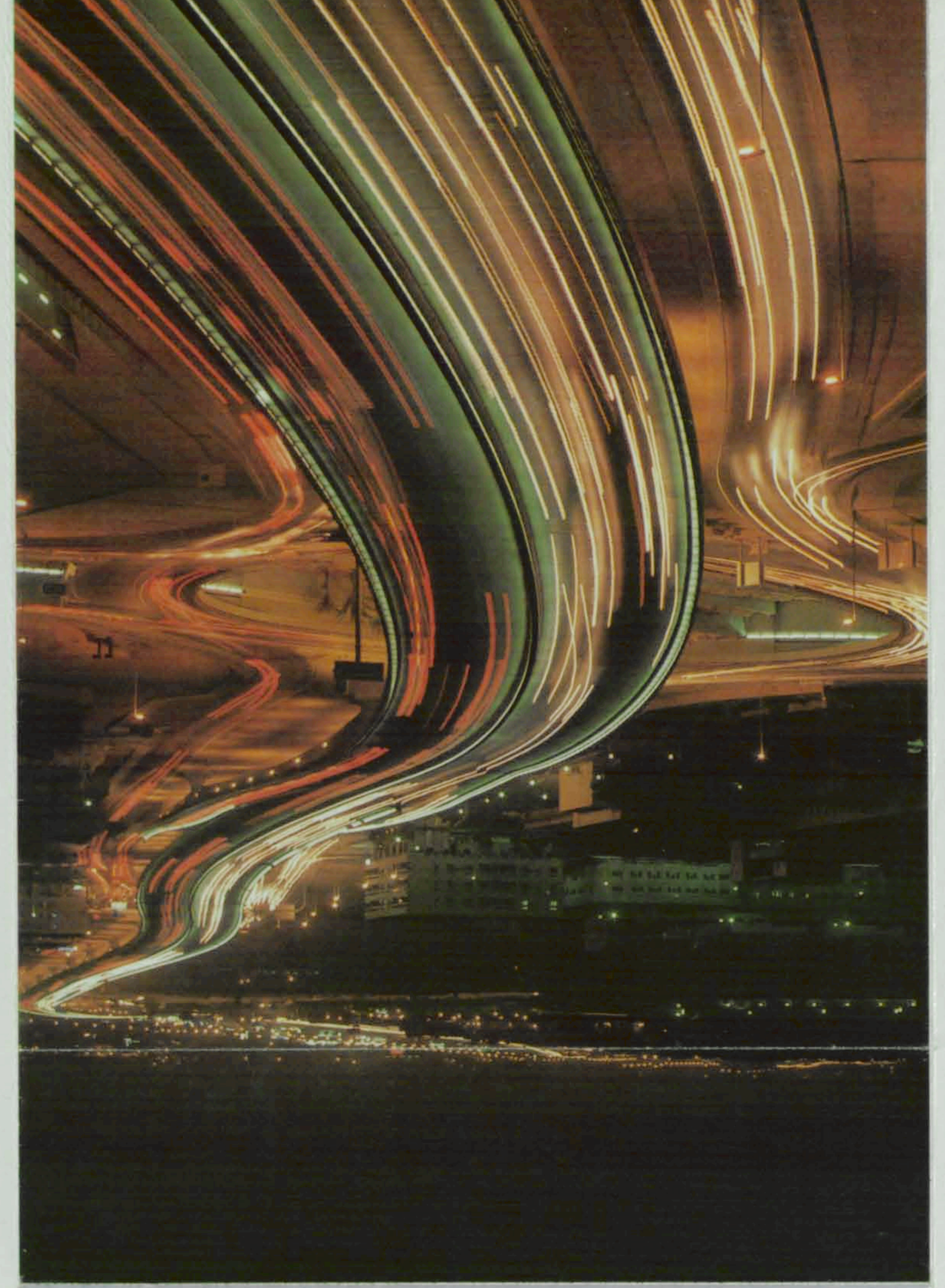
Gulf Street Viaduct in Riyadh.

1406 H. 1986 A.D. ١٤٠٦هـ ١٩٨٦م



HIS MAJESTY KING FAHD BIN ABDUL AZIZ

HIS ROYAL HIGHNESS PRINCE ABDULLAH BIN ABDUL AZIZ



FOREWORD

It is with great pleasure that I present this brochure to you. In a few pages, it depicts a fraction of the progress the Kingdom of Saudi Arabia has achieved under wise and prudent leadership.

We must acknowledge that a brochure such as this cannot provide a comprehensive picture of all the Kingdom's accomplishments in the transportation sector. We are able to provide only a glimpse of some of the achievements in this area.

In a large and sparsely populated country such as Saudi Arabia which covers over two and a half million square kilometers, a well-planned road systems plays a vital role in contributing to over-all development.

In addition to the vastness of the Kingdom, Saudi Arabia is characterized by extremes of climate and geography ranging from high, rugged mountain ranges to rocky plains to vast deserts of sand. In carrying out its vital assignment, the Ministry of Communications has had to deal with the complex problems caused by such extremes.

It is our goal to link all of the Kingdom's population centers with a first class road system of about 30,000 kilometers of paved roads. Under the direction of the King and Crown Prince we have strived for continued technological advancements to aid the country in reaching its development goals.

Although the Ministry of Communications has accomplished a great deal in just a few years, we are placing the utmost importance on continuing technological advances in developing our entire transportation system.

May God grant us success.

Minister of Communications
Hussein Al-Mansouri

Sha'ar Descent in Asir Province.

Night view of Hijas Interchange in Riyadh.

THE MINISTRY OF COMMUNICATIONS

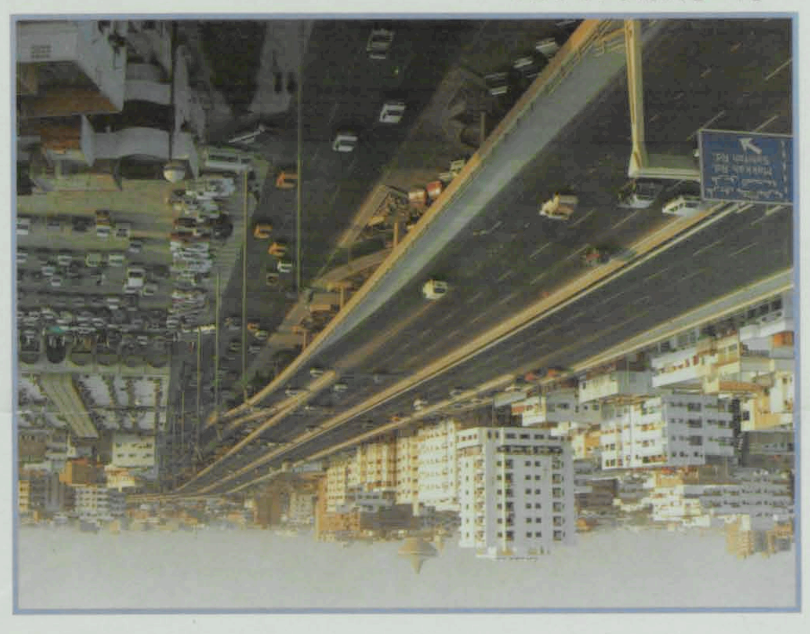
The Ministry of Communications came into existence during the early days of the Kingdom of Saudi Arabia. The founder of the country, H.M. the late King Abdul Aziz Al-Saud, recognized the role that transportation and communications would play in the development and progress of the newly born Kingdom.

In keeping pace with the progress achieved in other sectors in the Kingdom, the transportation and communications sector has witnessed amazing changes during the last few decades. As the material resources of the Kingdom have grown in recent years, the role of transportation and communications has continued to expand. This sector has played a key role in the over-all development of the Kingdom's economy and has had a significant impact on the lives of the Kingdom's citizens.

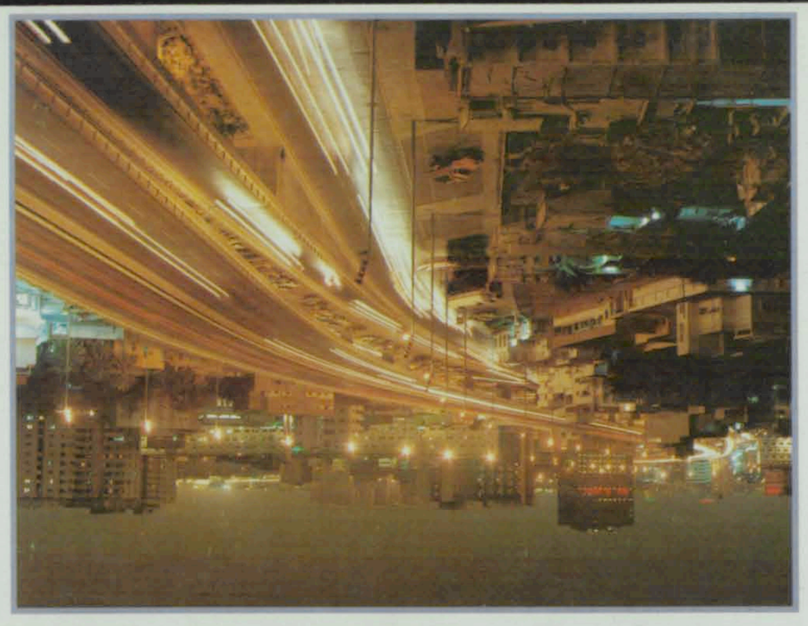
In 1395 H. (1975 A.D.) the Ministry of Communications was reorganized and given responsibility for all roads, bridges and related structures. It was also given responsibility for the development and supervision of all land and marine transportation in the Kingdom.

The Ministry's task in building the Kingdom's transportation infrastructure was outlined under a series of 5-year plans which were carefully laid-out plans for the development of the Kingdom in all sectors. Based on the 5-year plans, the Ministry implemented its plans to develop the road network and other modes of transportation. The plans were implemented in accordance with a carefully prepared time schedule. As the Ministry gained experience over the years, it continually upgraded its performance standards to keep pace with the most advanced technology in the world.

Because of the magnitude of the task of developing a complete transportation infrastructure in just a few years, the Ministry has utilized the talents of a variety of specialized consultants and contractors. The firms, both domestic and foreign, have been used to expedite the design and construction of all major projects. The Ministry has encouraged the expansion of domestic Saudi companies as much as possible to develop the necessary expertise and technology within the Kingdom.



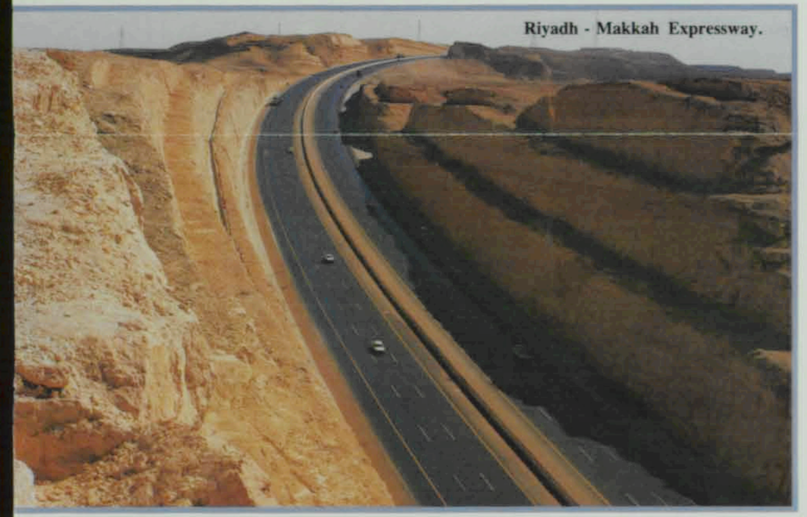
Prince Fahd Viaduct in Jeddah.



ROAD PROGRAMS

The Ministry of Communications has undertaken a comprehensive highway development program to keep pace with the dramatic increase in traffic throughout the Kingdom. In recent years the Ministry has focused increased attention not only on the quantity of the road network but also on the quality of the system. The present program emphasizes the following three objectives:

1. Expanding the road network to reach all the principal areas in the Kingdom and provide access to as many towns and villages as possible.
2. Widen existing roads and construct more dual carriageways and expressways in areas with traffic congestion.
3. Improve preventive maintenance services and road safety measures.



Riyadh - Makkah Expressway.

THE DEVELOPMENT PLANS

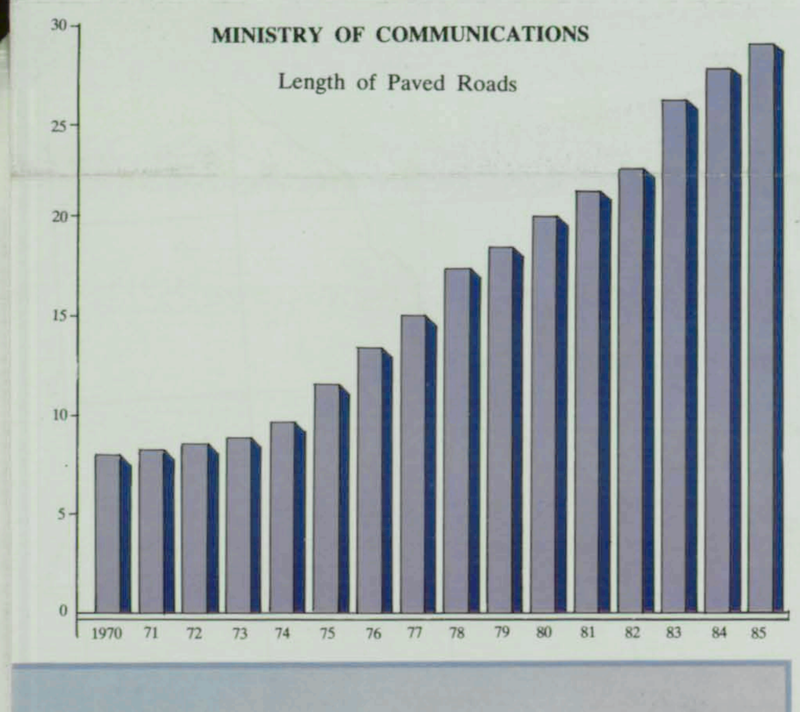
Ministry of Communications' projects have played a major role in the Kingdom's Five-Year Development Plans. The development of the road network and other key transportation elements are vital to the successful economic development of the Kingdom.

During the first plan 1390-1395 H. (1970-1975 A.D.) most of the districts and main cities were linked by a network of paved roads. By the end of this plan 1219 km. of asphalt roads and 8510 km. of agricultural roads had been built.

With the second development plan 1395-1400 H. (1975-1980 A.D.) the major projects had been completed. The total length of paved roads had grown to 21,583 km. and agricultural roads had increased to 24,186 km. Most of the Kingdom's districts were now connected by a modern road network.

By the end of the third development plan in 1405 H. (1985 A.D.) the Kingdom had 29,655 km. of paved highways. Agricultural roads had been increased to 50,655 km.

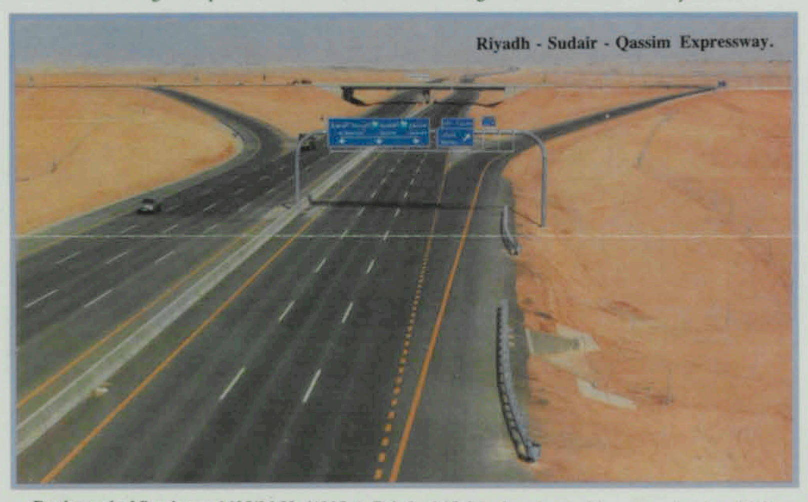
The fourth development plan has set a target of increasing the road network in the Kingdom to 16,000 km which will include 35,000 km of paved highways (primary, secondary and feeder roads) and ,000 of agricultural roads.



EXPRESSWAYS AND DUAL CARRIAGEWAYS

To keep pace with the Kingdom's rapid growth, the MOC has undertaken a major program to upgrade the national highway network. Where traffic volumes warrant, primary roads have been upgraded into expressways and dual carriageways (divided highways).

The expressways are designed to the most modern international standards and include 3 to 8 lanes in each carriageway with appropriate fencing, signals, directional and warning signs. Expressways also include interchanges to provide access to towns and villages and to enhance safety for users.



Riyadh - Sudair - Qassim Expressway.

By the end of fiscal year 1405/06 H. (1985 A.D.) the MOC had contracted for approximately 3500 km of dual carriageways and expressways. The most important are:

- Riyadh - Dammam Expressway • Riyadh - Sudair - Qassim Expressway
- Riyadh - Al-Kharj Dual carriageway • Makkah Al-Mukarramah - Madinah Expressway
- Makkah Al-Mukarramah - Jeddah Expressway

RING ROADS

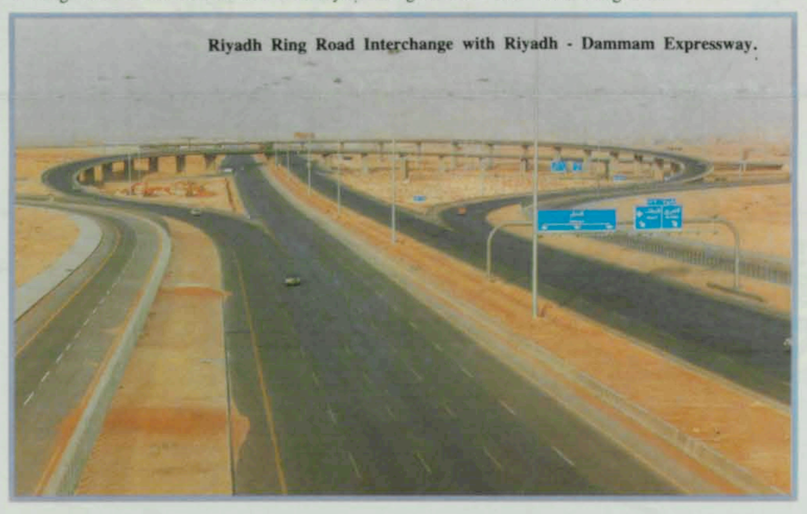
As a result of the rapid social and economic growth and development which has taken place in the Kingdom during recent years, the Kingdom's cities have expanded at an equally rapid rate. Many cities have grown so rapidly that in just a few years they have mushroomed to many times their original size. Traffic has grown just as rapidly and has exceeded all expectations leading to traffic congestion on major city streets throughout the Kingdom.

To alleviate this problem and facilitate the movement of traffic in the Kingdom's cities, the MOC has constructed ring roads around several major cities. The ring roads have been built to expressway standards and include service roads, interchanges, bridges and underpasses with accompanying lighting, landscaping and safety provisions.

The modern ring road concept envisions easy access to outlying areas without the hindrance of traffic lights and the other barriers of city streets. Ring roads also divert through traffic, especially heavy trucks, away from more congested city centers helping to alleviate noise and exhaust pollution. Finally, ring roads facilitate the more rapid movement between various sections of the city which is particularly important for public service vehicles such as fire-fighting equipment, ambulances, and police.

Some of the most important ring roads are:

- Riyadh Ring Road • Makkah Al-Mukarramah Ring Roads
- Ring Road II in Al-Madinah • Buraydah Ring Road • The Abha Ring Belt

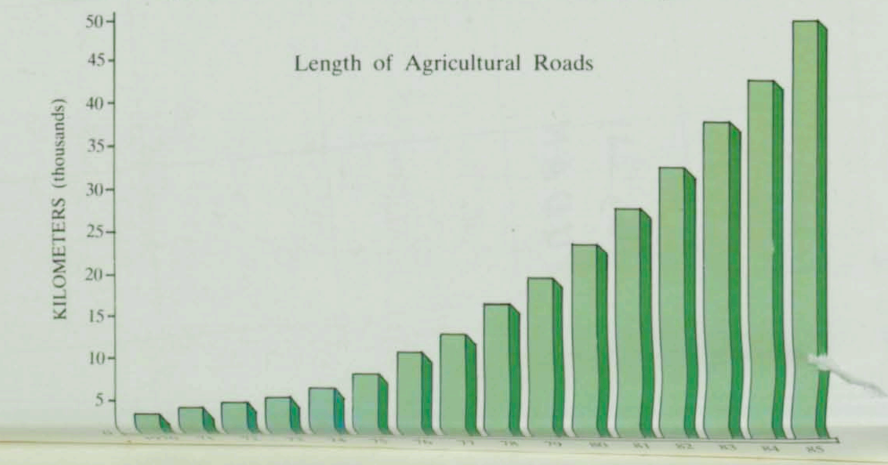


Riyadh Ring Road Interchange with Riyadh - Dammam Expressway.

AGRICULTURAL ROADS

In 1384 H. (1964 A.D.) the Ministry began a major effort to improve agricultural roads in the Kingdom. These roads are built to connect often isolated villages with the Kingdom's main road network and serve the vital purpose of improving access of agricultural areas to important markets throughout the Kingdom.

Sixty-five work teams have been charged with this task. Each team is provided with all necessary equipment as well as the related technical and administrative personnel. To date, an estimated 51,226 kilometers of agricultural roads have been built to connect 7,500 rural villages.

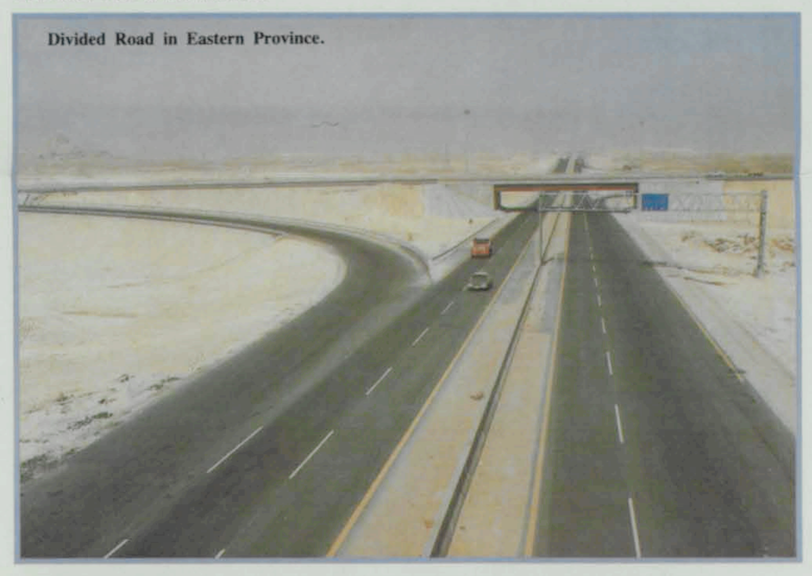


THE MAIN ROADS

Main roads are built by the Ministry of Communications with assistance from both Saudi and foreign consultants with experience and expertise in this area. This program is part of the Kingdom's overall development program affecting every aspect of life in the Kingdom.

The primary objective is to connect the various sections of each province and then the various provinces with each other. In addition the road network will connect the Kingdom with its neighbors in the Arab world.

By the end of fiscal year 1405/06 H. (1985/86 A.D.) approximately 30,000 kilometers of paved roads had been built in the network.



Divided Road in Eastern Province.

TIHAMA DESCENTS

The development of a modern and efficient road network is a basic prerequisite for the economic and social development of any country. The achievement of this goal was particularly arduous in the south-western region of Saudi Arabia. The villages and agricultural areas of the plains near the Red Sea were effectively cut off from the important markets of the interior plateau by the rugged Tihama Mountains. The mountains, which reach a height of over 3,000 meters, extend for over 500 kilometers in the south-western part of the Kingdom.

In order to overcome these enormous physical obstacles and facilitate transportation between the areas of Jizan, Abha, Najran, Jeddah and Riyadh, the Ministry of Communications undertook a feasibility study for a road network for the region. The study, begun in 1394 H. (1974 A.D.), led to the selection of twelve mountain road projects, or descents, for design and construction.

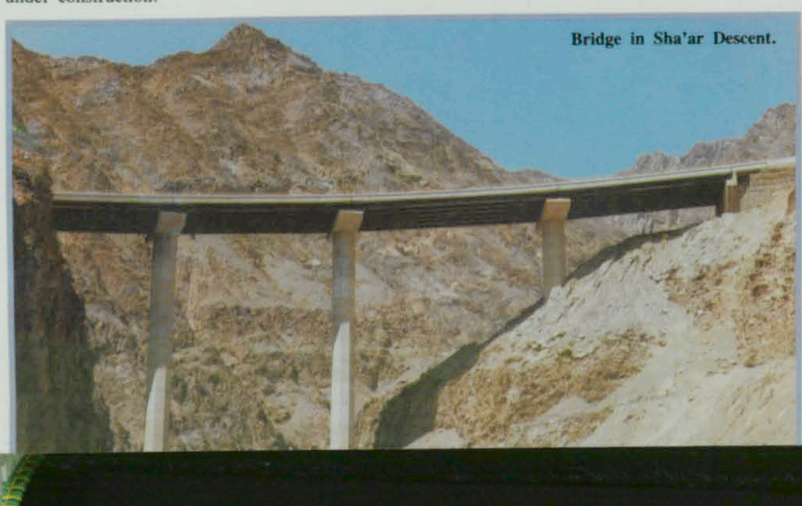
Construction on the project began in 1397 H. (1977 A.D.). The Sha'ar and Al-Jowa descents have already been completed while work on the Dhala'a and Al-Baha descents is currently nearing completion. Engineering studies and design work on the remaining Tihama descents have been completed.

Sha'ar Descent in Asir Province.

BRIDGES

In addition to the other major accomplishments of the Ministry of Communications, a significant bridge building program has been undertaken. Thus far, more than 4,200 bridges have been built in the Kingdom.

As a result of a thorough review of all existing bridges, the Ministry's Bridge Section has developed updated bridge specifications to reflect the most advanced technology. Load capacity of bridges and box culverts will be upgraded to 60 tons. These advanced specifications are being applied to all projects under construction.



Bridge in Sha'ar Descent.

MAINTENANCE

After having completed a substantial portion of the Kingdom's road network, the Ministry has begun to focus increased attention on maintaining the roads, bridges, and other structures and safety features.

Maintenance is of two types. The first is routine maintenance which includes the repair of normal wear and tear as well as cleaning of road surfaces, repairing signs, and repainting as necessary. The second is preventive maintenance aimed at the protection and upkeep of roads, bridges and related structures. This may include reconstruction and upgrading of roads where necessary to meet changing conditions.

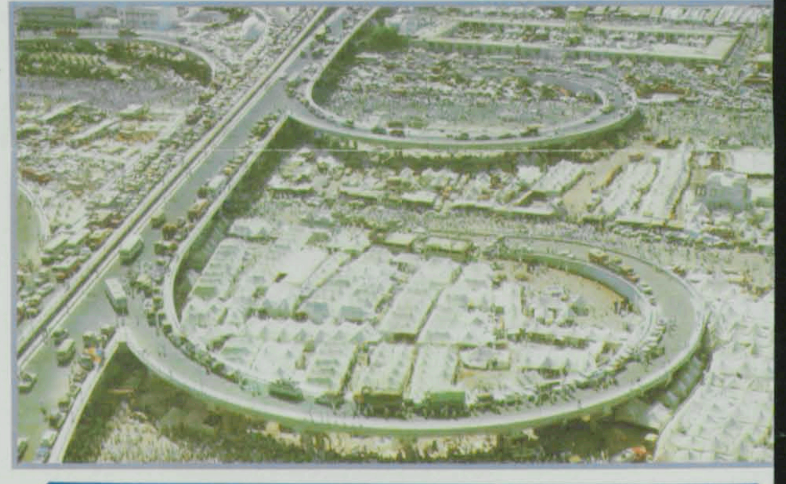
SPECIFICATIONS

The Kingdom is characterized by great extremes of climate and topography ranging from rugged mountains to flat, rocky plains to vast expanses of sand desert. To best meet the requirements dictated by the Kingdom's physical environment, the most modern international specifications have been adapted for highway design and construction in Saudi Arabia. The Ministry has published the general requirements as well as detailed specifications for the use of consultants and contractors engaged in designing and building the road network for the Kingdom.

Al-Madinah Road in Jeddah.

SERVICES FOR PILGRIMS

The Ministry of Communications has given the highest priority to provide improved services to the millions of pilgrims who annually visit the Holy Places in the Kingdom. A unique network of roads, bridges and tunnels for both vehicles and pedestrians has been constructed to assist the pilgrims.



Roads and Pedestrian Facilities in the Holy Places.

CIVIL AVIATION – AIRPORTS

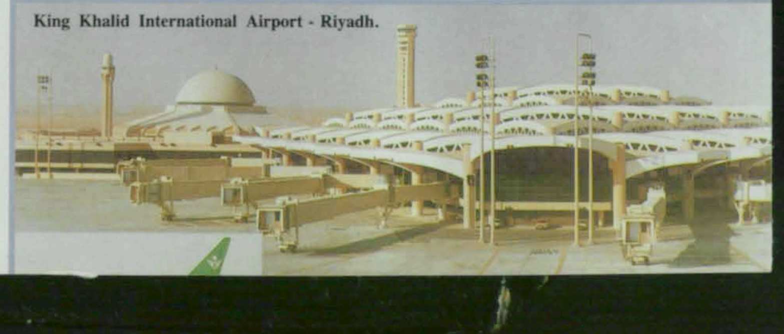
There are currently 23 domestic and international airports in the Kingdom. The three main international airports are in Jeddah, Riyadh, and Dammam.

King Abdul Aziz International Airport in Jeddah, with a land area of 105 square kilometers, was opened by King Khalid in April 1981. Riyadh's King Khalid International Airport was opened by King Fahad in November 1983. It has a total land area of 225 square kilometers. The third, King Fahd International Airport in Dammam is currently under construction and will replace the existing international airport in the Eastern region.

All three airports are among the largest airport projects ever undertaken and are under the supervision of the Presidency of Civil Aviation – International Airports Projects.

SAUDI ARABIAN AIRLINES

Saudia, the national flag carrier, now operates a fleet of 95 aircraft including Lockheed Tristars, Airbuses and the most advanced Boeing 747's. Saudia provides service to 23 domestic airports and to international destinations. The airline currently carries over 11 million passengers annually and has a workforce of over 10,000.



King Khalid International Airport - Riyadh.

5.3 - Traffic growth

The increase in vehicle numbers on the roads has risen sharply in the last ten years and the trend is still upward. Statistical figures are available from the year 1971, which is considered as the base year for traffic growth counts. No previous accurate figures are available. Every major city in the Kingdom has seen dramatic increases in the number of vehicles on the roads which has led to periods of acute congestion and a high frequency of accidents. For example the records of the number of vehicles in use in the city of Jeddah show that it has increased fifty-two times from 1971 to 1981. In 1971, the number of vehicles was 13,217 and it reached 690,073 in 1981. Similarly every major city has experienced a rapid increase in the number of vehicles, in some cases higher than the example given above. Table 5.3.1 shows the total increase of differing types of vehicles in Saudi Arabia between 1971 and 1985. Vehicles are classified as heavy vehicles, passenger cars and taxis, motor-cycles and buses. The number of vehicles registered in 1971 were considered as the base year with which to compare the increase in following years. As shown in table (5.3.1), the number of vehicles has risen very sharply from 144,768 in 1971 to above four million in 1985 (125).

TYPE / YEAR	Heavy Vehicles		Passenger Cars Private		Passenger Cars Taxi		Buses		Motor-Cycle		Total	
	Number	Incr. %	Number	Incr. %	Number	Incr. %	Number	Incr. %	Number	Incr. %	Number	Incr. %
1971	65541	100*	61541	100*	14260	100*	3426	100*	-	-	144768	100*
1972	85812	131	73876	120	16103	113	4394	128	-	-	180185	124
1973	118451	181	96766	157	22054	155	5703	166	-	-	242974	168
1974	168730	257	144340	235	33887	238	8065	235	-	-	355022	245
1975	253077	386	209379	340	42401	297	9504	277	-	-	514361	355
1976	389648	595	313267	509	58916	413	12612	368	-	-	774443	535
1977	571874	873	446984	726	78160	548	15955	466	-	-	1112973	769
1978	736356	1124	583421	948	94234	661	18888	551	-	-	1432909	990
1979	852035	1300	742703	1207	107013	750	21365	624	-	-	1723116	1190
1980	994458	1517	939732	1527	108033	758	25778	752	1478	100	2069479	1430
1981	1169581	1785	1155508	1878	108033	758	29942	872	4839	327	2467903	1705
1982	1407745	2148	1463413	2378	108033	758	32493	948	7127	482	3018811	2085
1983	1669223	2547	1748365	2841	108033	758	35077	1024	8311	562	3569009	2465
1984	1826506	2787	1932921	3141	109409	767	38796	1132	12239	828	3919871	2708
1985	1925507	2938	2053918	3337	111757	784	40664	1187	12399	839	4144245	2863

Table5.3.1 Vehicle type growth in Saudi Arabia between 1971 and 1985.(Ref. No.125)

*Note: number of cars registered in 1971 is considered as the base year, 100 per cent.

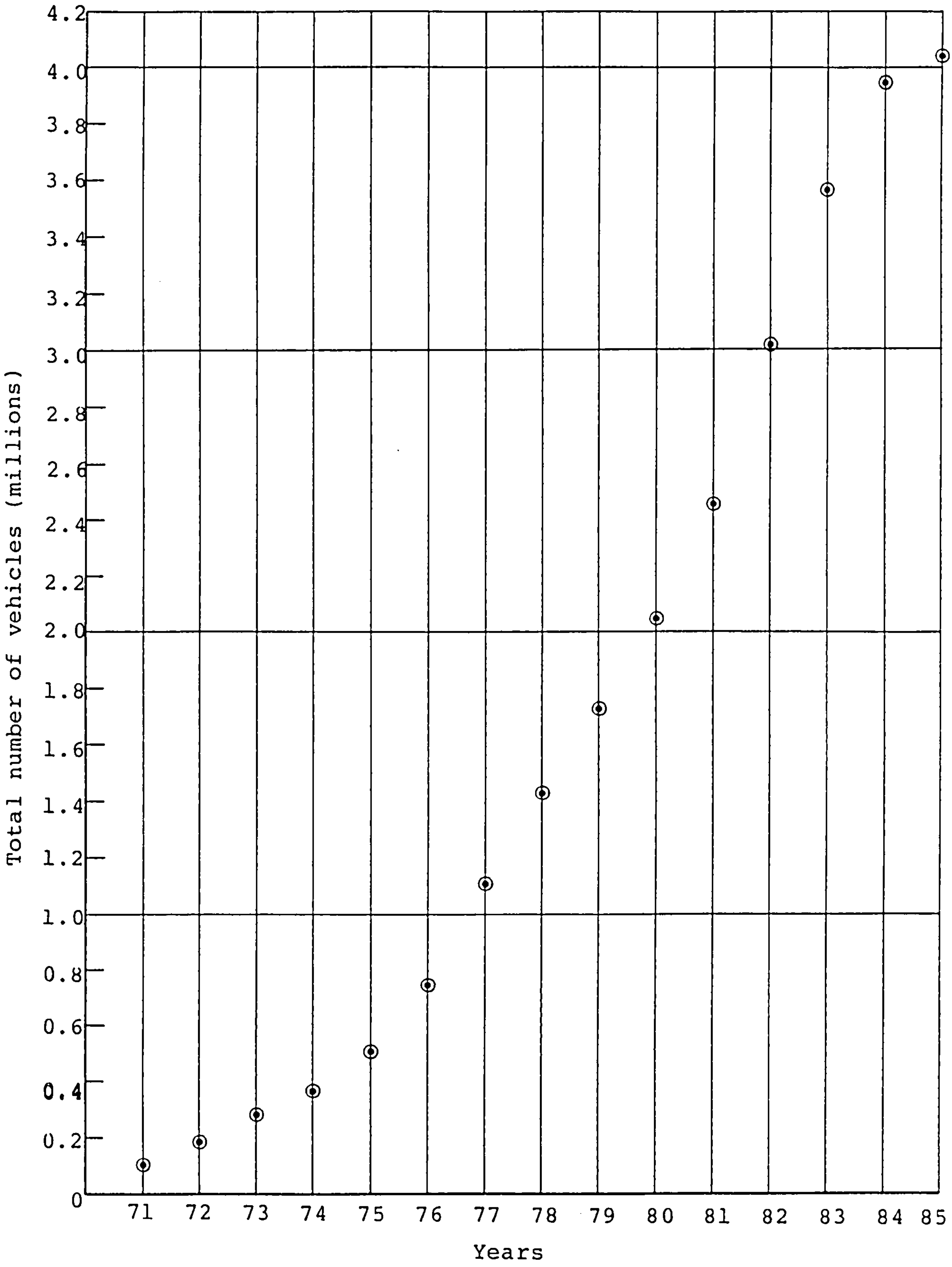


Figure 5.3.1 Vehicle Growth in Saudi Arabia.

Car ownership in Saudi Arabia was estimated as 0.2 cars per person in 1980 and 0.44 cars per person in 1985, excluding heavy commercial vehicles (6). The vehicle type increase shown in table (5.3.1) is represented graphically by figure (5.3.1). Heavy commercial vehicles and buses show high percentage increases and represent 47.44% of the total number of vehicles registered in 1985.

5.4 - Effects of vehicle type

5.4.1 - Vehicle type classification

It is only recently that the Ministry of Communications for the Kingdom, with the collaboration of some Universities, have begun to investigate the problems caused over the last two decades by the lack of controls on imported vehicles. Vehicle dealers have brought in heavy commercial vehicles from all over the world, with differing specifications, and there have been no government controls of size, weight or type. Furthermore, in the absence of operating rules, vehicles designed to carry 40 tons have been carrying weights of up to 100 tons, with dire effects on roads.

The University of Petroleum and Minerals carried out a study, supervised by the National Centre for Science and Technology and sponsored by the Ministry of Communications. The Canadian classification of heavy commercial vehicles was recommended as the one to adopt, see figure (5.5.1-1) and table (5.5.1-1) and the study included interviews with local authorities, field studies of the physical and operational characteristics of vehicles; speed limits; and driver characteristics and public obedience to traffic control devices.

Canadian size and weight requirements for commercial vehicles

Revised September 1984. Note however, that the regulations change frequently. Contact the appropriate authority at the number listed at the bottom of the chart for current information. Most, particularly B.C. and N.B., handle enquiries in metric only

	NFLD.	N.S.	N.B.	P.E.I.	QUE.	ONT.	MAN.	SASK.	ALTA.	B.C.	Y.T.	N.W.T.
LENGTH Single Powered Vehicle (ft, m)	41.0 12.5	41.0 12.5	41.0 12.5	40.0 12.2	41.0 12.5 KK	41.0 12.5	41.0 12.5	41.0 12.5	41.0 12.5	41.0 12.5	41.0 12.5	40.0 12.2
OVERALL LENGTH Combination (ft, m)	68.9 21.0	68.9 21.0	65.6 20.0 EE	69.0 21.0	68.9 21.0	75.5 23.0	65.6 20.0 D	65.6 20.0 D	65.5 20.0 D	65.6 20.00 CC	65.6 20.0 NN	80 24.4
MAXIMUM WIDTH (in., m)	102 2.6	102.4 2.6	102.4 2.6	102.4 2.6	102.4 2.6	102.4 2.6	102.4 2.6	102.4 2.6	102 2.6	102.4 2.6	102.4 2.6	120.0 3.05
MAXIMUM HEIGHT (ft, m)	13.6 4.15	13.6 4.15	13.5 4.12	14.8 4.5	13.6 4.15	13.6 4.15	13.6 4.15	13.6 4.15	13.5 4.15	13.6 4.15	13.8 4.2	13.5 4.11
NUMBER OF TRAILERS ALLOWED	2	2,S	2	A	2	A,S	2,A,Q	2,A	2,A	2,B	2,W	2,KK
MAXIMUM LOADS (lb, kg) — Single Axle (except front)	20,000 9 100	19,900 9 000 F	19,900 9 000	20,000 9 100	22,000 10 000 J	22,000 10 000 Z, MM	20,000 9 100 X	20,000 9 100 K	20,000 9 100	20,000 9 100 DD	22,000 10 000	18,000 8 165
MAXIMUM LOADS (lb, kg) — Tandem Axles	39,700 18 000	37,500 17 000 F	39,700 18 000 M	40,000 18 100 K,M	44,000 20 000 J	42,100 19 100 AA	35,000 16 000 X	35,000 15 600 K	35,000 16 000	37,500 17 000 C	42,100 19 100 E	36,000 16 330
Straight truck: 2 axles	30,000 13 500 L	29,800 13 500 L	29,800 13 500 FF	30,000 13 650 L	40,800 18 500 J,P	41,800 19 000 G,Z	32,200 14 600 X	32,000 14 500 K	36,100 16 400	32,000 14 600 T	33,000 15 000 E	26,460 12 000 E
Tandem straight truck: 3 axles	47,400 21 500 L,C	47,400 21 500 L	47,400 21 500 FF	47,500 21 550 L	62,800 28 500 J,P	62,000 28 100 G,Z	47,400 21 500 X	47,000 21 500 K,JJ	51,400 23 320	49,600 22 500 C,T	55,100 25 000 E	45,200 20 500 E
Tractor and semitrailer: 3 axles	50,000 22 500 L	49,600 22 500 L	49,600 22 500 FF, BB	50,000 22 550 L	62,800 28 500 J,P	63,900 29 000 G,Z	52,250 23 700 X	52,000 23 600 K,JJ	56,000 25 400 W	52,250 23 700 T	55,100 25 000 E	45,200 20 500 E
Tractor and tandem semitrailer: 4 axles	69,700 31 600 L,C	69,500 31 500 L,W	69,500 31 500 FF,BB	70,000 31 800 K,L,M,	84,900 38 500 J,P	84,000 38 100 G,Z	67,500 30 600 X	67,000 30 400 JJ,K	71,000 32 200 W	69,700 31 600 C,T	75,200 34 100 E	62,830 28 500 E
Tandem tractor and semitrailer: 4 axles	67,500 30 600 L,C	67,200 30 500 L,W	67,200 30 500 FF,BB	67,500 30 700 L,M	84,900 38 500 J,P	84,000 38 100 G,Z	67,500 30 600 X	67,000 30 400 JJ,K	71,000 32 200 W	69,700 31 600 C,T	77,200 35 000 E	62,830 28 500 E
Tandem tractor and tandem semitrailer: 5 axles	87,100 39 500 L,C	87,100 39 500 L,W	87,100 39 500 FF,BB	87,500 39 700 K,L,M	106,900 48 500 J,P	104,000 47 200 G,Z	82,700 37 500 X	82,000 37 500 K,JJ	86,000 39 000 W	87,000 39 500 C,S,T,V	95,250 43 200 E	80,470 36 500 E
Tandem tractor and 3-axle semitrailer: 6 axles	107,000 48 500 I,L,C	106,900 48 500 L,W	106,900 48 500 FF,BB	107,500 48 700 K,L,M	126,800 57 500 J,P	125,000 56 700 G,Z	82,700 37 500 X	82,000 37 500 K,JJ	86,000 39 000 W	95,900 43 500 C,S,T,U,V	116,200 52 700 E	89,290 40 500 E
Straight truck and full trailer: 4 axles	69,500 31 500 L	69,500 31 500 L	69,500 31 500 FF,BB	Special permit required	84,900 38 500 J,P	86,000 39 000 G,Z	72,300 32 800 X	72,300 32 800 JJ,K	76,000 34 500 W	72,300 32 800 T	77,200 35 000 E	62,830 28 500
Straight truck and tandem full trailer: 5 axles	89,300 40 500 L,C	89,300 40 500 L,W	89,300 40 500 FF,BB		106,900 48 500 J,P	106,000 48 100 G,Z	87,500 39 700 X	87,000 39 500 K,JJ	91,000 41 300 W	83,800 38 000 C,T,V	90,600 41 100 E	80,470 36 500
Tandem straight truck and tandem full trailer: 6 axles	107,000 48 500 L,C	107,000 48 500 L,W	106,900 48 500 FF,BB		126,800 57 500 J,P	126,000 57 200 G,Z	102,750 46 600 X	102,000 46 300 K,JJ	106,000 48 100 W	107,100 48 600 T,V	117,300 53 200 E	89,290 40 500
Tractor semitrailer, full trailer: 5 axles - A-train	87,100 39,500	89,300 40 500 L,W	Special permit required		106,900 48,500 J,P	108,000 49 000 G,Z	92,400 41 900 X	92,000 41 700 K,JJ	96,000 43 500 W	83,800 38 000 T,V	Special permit required.	89,290 40 500
Tandem tractor, tandem trailer, full trailer: 7 axles - A-train	115,700 52,500	110,250 50 000 W			126,800 57 500 J,P	140,000 63 500 G,Z	122,800 55 700 X	118,000 53 500 K,JJ	118,000 53 500 W	127,200 57 700 C,T,V		116,840 53 000
Tandem tractor, tandem semitrailer, tandem full trailer: 8 axles - A-train	118,000 53 500	110,250 50 000 W			126,800 57,500 J,P	140,000 63 500 G,Z	124,600 56 500 X	118,000 53 500 K,JJ	118,000 53 500 W	140,000 63 500 C,T,V,		116,840 53 000
Tandem tractor, tandem semitrailer, tandem semitrailer: 7 axles - B-train	124,600 56 500 H,L,LL	110,250 50 000	125,000 56 500 BB,FF	118,000 53 500	126,800 57 500 J,P	140,000 63 500 G,Z	118,000 53,500 X	118,000 53 500 K,JJ	118,000 53 500 W	124,600 56 500 C,T,V	139,600 63 300 G	116,840 53 000
Tandem tractor, 3 axle semitrailer, tandem semitrailer: 8 axles - B-train	124,600 56 500 H,L,LL	110,250 50 000	Special Permit	118,000 53 500	126,800 57 500 J,P	140,000 63 500 G,Z	118,000 53 500 X	118,000 53 500 K,JJ	118,000 53 500 W	140,000 63 500 C,T,U,V,	139,600 63 300 G	116,840 53 000
For further information contact:	709/737-3808	902/424-5973	506/453-2802	902/892-5306	418/643-6864	416/248-7154	204/945-3898	306/565-4848	403/340-5293	604/387-4404	403/667-5670	403/920-8633

Explanatory notes

BUS & TRUCK TRANSPORT

Figure 5.4.1-1 Canadian legal size and weight for heavy commercial vehicles.
(Reproduced from reference No.123)

VEHICLE SIZES & WEIGHTS — MAXIMUM LIMITS — JANUARY 1, 1984

DES = Interstates and federally designated state highways
OTHER = All other state highways and supplemental routes

[illegible]

	LENGTH (FEET)										HEIGHT (FEET)	WIDTH (INCHES)		WEIGHT (1,000 POUNDS)					
	INTERSTATE AND DESIG HWYS (DES)					STATE AND SUPP HWYS (OTHER)								Gross Axle Weight		Tandem Axle Weight		Gross Vehicle Weight	
	Straight Trucks	Combinations †		Trailing Units ‡		Straight Trucks	Combinations †		Trailing Units ‡										
		Tractor Semi-Trailer	Tractor Twin Trailers	Semi-Trailer	Trailer		Tractor Semi-Trailer	Tractor Twin Trailers	Semi-Trailer	Trailer		DES	OTHER	INT	Other	INT	Other	INT	Other
ALABAMA *	40	[illegible]	[illegible]	48	28	[illegible]	[illegible]	A	NS	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
ALASKA *	40	65	70	[illegible]	NS	[illegible]	65	[illegible]	[illegible]	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
ARIZONA	40	[illegible]	[illegible]	[illegible]	28.5	[illegible]	[illegible]	[illegible]	[illegible]	28.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
ARKANSAS	40	[illegible]	[illegible]	48	28.5	[illegible]	[illegible]	[illegible]	48	28.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
CALIFORNIA	40	B	B	B	B	[illegible]	[illegible]	[illegible]	B	B	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
COLORADO	35	[illegible]	[illegible]	[illegible]	[illegible]	35	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
CONNECTICUT	60	[illegible]	[illegible]	48	28	[illegible]	[illegible]	[illegible]	48	28	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
DELAWARE	40	[illegible]	[illegible]	53	28	[illegible]	[illegible]	65	NS	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
D C	40	[illegible]	[illegible]	48	28	[illegible]	55	A	NS	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
FLORIDA	F	[illegible]	[illegible]	48	28	F	55	A	48	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	44	[illegible]	[illegible]
GEORGIA	60	[illegible]	[illegible]	48	28	60	[illegible]	A	[illegible]	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
HAWAII	40	NS	NS	NS	NS	40	[illegible]	65	NS	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
IDAHO	40	[illegible]	[illegible]	48	[illegible]	40	[illegible]	[illegible]	48	[illegible]	14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
ILLINOIS	42	[illegible]	[illegible]	[illegible]	28.5	42	G	G	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
INDIANA	36	[illegible]	[illegible]	[illegible]	28.5	36	[illegible]	[illegible]	[illegible]	28.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]
IOWA	40	[illegible]	[illegible]	53	28.5	[illegible]	60	[illegible]	NS	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
KANSAS	42.5	[illegible]	[illegible]	[illegible]	28.5	42.5	[illegible]	[illegible]	[illegible]	28.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
KENTUCKY	45	[illegible]	[illegible]	48	28	45	[illegible]	65	NS	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
LOUISIANA	40	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	A	50	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
MAINE	45	[illegible]	[illegible]	48	28	45	[illegible]	A	45	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
MARYLAND	40	[illegible]	[illegible]	48	28	[illegible]	55	65	NS	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
MASSACHUSETTS *	35	60	NS	45	A	35	[illegible]	NS	45	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
MICHIGAN	40	[illegible]	[illegible]	48	28.5	[illegible]	NS	59	48	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
MINNESOTA	40	[illegible]	[illegible]	48	28.5	[illegible]	65	A	48	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
MISSISSIPPI	35	[illegible]	[illegible]	[illegible]	[illegible]	35	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]
MISSOURI	40	[illegible]	[illegible]	48	28	[illegible]	[illegible]	[illegible]	NS	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
MONTANA	40	[illegible]	[illegible]	48	28.5	40	[illegible]	[illegible]	48	28.5	13.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]
NEBRASKA	40	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	53	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]
NEVADA	40	[illegible]	[illegible]	48	28.5	[illegible]	70	75	NS	NS	14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
NEW HAMPSHIRE	35	[illegible]	[illegible]	48	28	35	[illegible]	[illegible]	48	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
NEW JERSEY *	35	[illegible]	[illegible]	48	28.5	35	55	A	NS	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
NEW MEXICO	40	[illegible]	[illegible]	48	28.5	40	65	65	NS	NS	13.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
NEW YORK	35	[illegible]	[illegible]	48	28.5	35	AA	65	AA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
NORTH CAROLINA	F	[illegible]	[illegible]	48	28	F	55	A	45	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	34	[illegible]	[illegible]
NORTH DAKOTA	50	[illegible]	[illegible]	NS	NS	50	75U	75U	NS	NS	13.5	[illegible]	[illegible]	[illegible]	[illegible]	34	34	80	[illegible]
OHIO *	40	[illegible]	[illegible]	48	28	40	60	65	NS	NS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	34	[illegible]	[illegible]
OKLAHOMA	40	[illegible]	[illegible]	[illegible]	29	[illegible]	[illegible]	[illegible]	[illegible]	29	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	34	[illegible]	[illegible]
OREGON	40	[illegible]	[illegible]	48	N	40	N	N	N	N	14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]
PENNSYLVANIA	40	G	[illegible]	[illegible]	28	[illegible]	60	A	NS	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
RHODE ISLAND *	40	[illegible]	[illegible]	48	28	40	60	A	NS	A	[illegible]	102	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	80	[illegible]
SOUTH CAROLINA *	F	[illegible]	[illegible]	48	28	F	60	A	45	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
SOUTH DAKOTA	45	[illegible]	[illegible]	[illegible]	S	45	[illegible]	[illegible]	NS	NS	13.5	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]	[illegible]
TENNESSEE	40	[illegible]	[illegible]	48	28	[illegible]	[illegible]	[illegible]	48	28	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]	[illegible]
TEXAS *	45	[illegible]	[illegible]	[illegible]	[illegible]	45	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	34	[illegible]	[illegible]
UTAH	45	[illegible]	[illegible]	48	[illegible]	45	[illegible]	[illegible]	48	[illegible]	14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]
VERMONT *	60	[illegible]	[illegible]	48	28	60	[illegible]	A	NS	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
VIRGINIA	40	[illegible]	[illegible]	[illegible]	28	[illegible]	[illegible]	A	[illegible]	A	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]	[illegible]
WASHINGTON	40	[illegible]	[illegible]	48	[illegible]	[illegible]	[illegible]	[illegible]	48	[illegible]	14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
WEST VIRGINIA	F	[illegible]	[illegible]	48	28	F	55	A	NS	A	[illegible]	102	[illegible]	[illegible]	[illegible]	34	34	[illegible]	[illegible]
WISCONSIN	40	[illegible]	[illegible]	48	28	[illegible]	60	A	48	A	13.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	[illegible]	[illegible]
WYOMING *	60	[illegible]	[illegible]	50	[illegible]	60	85	85	NS	NS	14	102	102	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

NOTE: No state shall prohibit the use of trailers or semitrailers of such dimensions as those that were in actual or lawful use in such state on Dec. 1, 1982. Neither shall any state prohibit the use of existing trailers or semitrailers of up to 28.5 feet in length in a truck tractor-semitrailer-trailer combination if those trailers and semitrailers were actually and lawfully operating on Dec. 1, 1982 within a 65-foot length limit in any state.

TOLERANCES
HAWAII — 5% weight tolerance on state and supplemental routes only.
KENTUCKY — 5% tolerance on length.
NEW HAMPSHIRE — 5% weight tolerance.
PENNSYLVANIA — 3% weight tolerance on axles and GVW of 73,280 pounds or less off interstate system; no tolerances on interstate system.

* We have not yet received primary information from these states. Limits shown are based on secondary sources or information published previously by them.

† Only tractor semitrailer and tractor twin-trailer combinations are considered here. For other combinations, contact state agency.

‡ Semitrailer in tractor semitrailer combination, and trailer in tractor-twin-trailer combination.

O No overall length restrictions imposed.

NS Not specified.

A. Not allowed (allowed in some states by permit).

B. On any hwy, tractor-semitrailer combo 65 if semitrailer exceeds 48 (distance between kingpin and rearmost semitrailer axle must be 38 or less on semitrailers exceeding 48).
On federally designed hwys, no overall combination length limitation or kingpin restriction if semitrailer is 48 or less.
Twin-trailer combinations 65 on all hwys if either trailer exceeds 28; 75 on non-designated hwys if either trailer is 28 or less, and unlimited length on federally designated system if either trailer is 28 or less.

C. 102 on all federal and state routes; 96 on all others.

D. Combinations with semitrailers or twin-trailers over 48 and 28 respectively may not exceed 70.

E. On class I, II and III hwys.

F. 2 axles 35; 3 axles 40.

G. Any semitrailer operated on any hwy whose length exceeds 48 is limited to a maximum distance of 42 from kingpin to center of rearmost axle. On class II, III, and non-designated hwys., maximum tractor-semitrailer wheel base 55. On class II hwys., maximum tractor-twin-trailer wheel base 65. On class III and non-designated hwys., maximum combination vehicle length 60.

H. 102 on class I and II hwys; 96 on class III and non-designated hwys.

J. 80,000 lbs on class AAA hwys; 62,000 lbs on class AA hwys; and 44,000 lbs on class A hwys.

K. 13.5 on designated hwys; 11.5 on all others.

L. 102 on hwys of 12 or more wide; 96 maximum allowed on all others.

M. Single Axle 22,400 lbs if GVW is less than 73,280 lbs and 20,000 lbs if GVW is greater than 73,280 but less than 80,000 lbs.
Tandem axles 36,000 lbs if GVW is less than 73,280 lbs and 34,000 lbs if GVW is greater than 73,280 but less than 80,000 lbs.

N. Tractor-semitrailer combo 60 for Group 1 hwys; 50 for groups 2 and 3 hwys. Semitrailers not specified for group 1; 40 for group 2 and 35 for group 3. Tractor-twin-trailers 75 for group 1, 65 for group 2 and 50 for group 3. Trailers 40 for group 1, 33 for groups 2 and 3.
On interstate and designated hwys no semitrailer or trailer in a twin trailer combo may exceed 40; both trailing units together measured from the front of the 1st to the rear of the 2nd may not exceed 68.

P. Any truck combination exceeding 73,280 lbs. GVW must not exceed 20,000 lbs. per single axle and 34,000 lbs. per tandem. Same requirements apply to any truck or combo more than 55 regardless of gross weight. Trucks 55 feet or less in length and not exceeding 73,280 lbs. must display a permit sticker. A tandem axle weight of 40,680 lbs. will be allowed and will not be subject to the bridge formula. Such trucks must have 1983 Georgia license tags.

R. Single Axle 22,000 lbs. if GVW is less than 73,280 lbs. and 20,000 lbs. if GVW is more than 73,280 lbs. but less than 80,000 lbs.

Q. No overall length limit if semitrailer is 48' or less; 60 overall length limit if semitrailer exceeds 48'.

S. In twin-trailer combos, semitrailer 48, trailer 28.5.

T. Semitrailer 48 if combo exceeds 60.

U. 2, 3 and 4-unit combos 110 on 4-lane divided hwys.

W. If axles of tandem are less than 8' apart.

X. 13.5 on interstate and designated hwys; 12.5 on non-designated routes.

Y. As measured from front of 1st trailing unit to rear of second.

Z. When GVW is 73,000 lbs. or less, single axle may not exceed 22,400 lbs. and tandem 36,000 lbs. If GVW exceeds 73,000 lbs., single axle may not exceed 20,000 lbs. and tandem 34,000 lbs.

AA. Tractor-semitrailer combo 60 if semitrailer is 45 or less. Tractor-semitrailer combo 55 if semitrailer is greater than 45 and less than 48.

Table 5.4.1-1 Maximum size and weight limits of heavy commercial vehicles. (Reproduced from reference No.123)

As a result of this and other studies by consultants, new regulations have been introduced by the Ministry of Communications for the maximum size of heavy commercial vehicles. These are:

Size	Length	18	metres
	Width	2.40	metres
	Height	4	metres

Weight	Maximum total weight	40 tons
	Maximum weight on single axle	13 tons
	Maximum weight on double axle	20 tons

To enforce these new measures, the Ministry of Communications decided to introduce a vehicle weighing system throughout the Kingdom. Thirty locations were selected mainly at ports and land entrances to the country. During 1985, twelve vehicle weighing-scales were constructed at ports, customs houses and at at entrances to newly-opened expressways.

5.4.2 - The effect of vehicle type on road network design

The transport sector of the Kingdom of Saudi Arabia is one of the most important sectors of the country's economy, since the economic development of the Kingdom relies heavily on its extensive road network. This network was designed according to AASHTO specifications and standards for a design life of 10 to 20 years.

It is well known to highway engineers that pavement damage increases exponentially with axle load. AASHTO road tests have shown that pavement damage is approximately proportional to the fourth power of the axle load so that if the axle load is doubled, damage to the pavement increases 16 times. Applying this logic to lighter loads of passenger cars, it can be seen that a fully-laden heavy commercial vehicle may cause approximately 10,000 times more damage than a passenger car..

The Kingdom's road network (road pavements and bridges) has suffered extensive damage both in rural and urban areas by overweight heavy commercial vehicles and is rapidly reaching a critical stage. Because of lack of enforcement many heavy commercial vehicle operators are loading their vehicles to the limit of the vehicle's structural capability, which may be two or three times the legal weight limit. Saudi Arabia has the most liberal weight allowances among Gulf countries and exceeds limits in Europe and the United States. The largest heavy commercial vehicles are currently limited to 40 tons gross weight with 13 tons on an individual axle, or 20 tons on a tandem axle. However, traffic police have many records of heavy vehicles weighing over 100 tons, with up to 32 tons on individual axles that are regularly traversing structures designed for 40 to 45 tons. As a result, the highway system is faced with premature bridge-deck failures and other structural problems. Similarly, the pavement is suffering damage from the same vehicles, with major rutting appearing on relatively new roadways. Another major effect of heavy commercial vehicles is damage to roadway directional signs. Heavy vehicles tend to exceed legal heights and widths which results in bringing down road signs, traffic signals and damaging the bottom of bridges. Many attempts to bring this problem under control have not been successful, primarily because of complaints from truck operators who have become accustomed to maximizing loads and view any restrictions as an unnecessary infringement on commerce.

The Ministry of Communications has undertaken many measures to protect the road network system;firstly through improving the design standards of pavement, and widening roadways and intersections; secondly by the construction of weigh stations throughout the Kingdom; and thirdly by attempting to reach an agreement with representatives from industry on modifications of weight and sizes of heavy commercial vehicles. Such discussions have helped the development of stricter specifications for heavy commercial vehicles in the Kingdom. Finally, the Ministry of Communications tries to control movements of heavy commercial vehicles, especially on urban roads by banning them from using certain routes through city networks using road signs and specifying entry to certain routes only during evening hours.

5.4.3 - The effect of vehicle type on urban traffic

Urban roads should be designed to be safe and to permit the free flow of traffic at a reasonable speed. Their traffic capacity should allow for projected demands in spite of the composition of the traffic which affects their performance and safety.

Because of the complexity of traffic problems in Saudi Arabia, design measures adopted by different local authorities are still inadequate to overcome these problems. This is mainly due to the lack of traffic engineering expertise in the local police and governmental authorities, the lack of traffic records, the lack of research efforts and the rapid change in both the highway system and vehicle growth. A further reason is the non-uniformity of traffic planning and standards, even in the same city where usually different foreign-based consultants are responsible for the design of the urban network.

Because of the lack of information about vehicle types, records concerning sizes, weights, operational characteristics and their percentage distributions throughout the Kingdom, the counter-measures adopted in many cases are often extreme, involving the use of the highest design standards for the reconstruction of roadways and urban intersections.

TEXT BOUND INTO

THE SPINE

ROAD	Section	Passenger cars %	Pick-ups Mini Bus %	Buses %	Motor-cycles %	Trucks 2 axle	Trucks 3 axle	Trucks 4 axle	Trucks Total Percentage	Grand Total
aidi	Start to End	55.48	21.85	0.37	1.77	13.02	2.80	6.65	22.47	100%
Port Road	Start to End	47.15	26.15	0.23	0.80	15.06	1.48	8.13	25.67	100%
dalus	Start to End	66.59	21.29	0.64	1.18	6.81	1.72	1.77	10.30	100%
Fahd	South to Palestine	72.24	23.79	0.11	1.78	1.89	0.19	•	2.08	100%
Fahd	North to Palestine	64.99	29.00	0.59	0.67	3.65	0.85	0.25	4.75	100%
njar Road	Start to End	52.80	32.55	0.79	1.51	9.45	0.75	2.15	12.35	100%
Abd Al Aziz Road	Start to End	86.74	5.36	3.76	3.86	0.28	•	•	0.28	100%
Street	Start to End	68.13	21.91	1.02	1.66	5.48	0.79	1.01	7.28	100%
t Access Road	Start to End	76.26	19.49	1.21	0.23	2.40	0.41	•	2.81	100%
l Road	Start to End	58.60	34.43	1.33	4.09	1.24	0.31	•	1.55	100%
aden Street	Start to End	48.00	43.60	0.60	1.95	5.25	0.30	0.30	5.65	100%
tine	Start to End	74.55	21.36	0.51	1.00	1.75	0.60	0.23	2.58	100%
kan	Start to End	57.70	31.49	0.30	2.20	6.60	0.40	1.20	8.20	100%
inal	Start to End	30.65	29.25	•	0.25	17.70	7.85	14.00	39.55	100%
che Road	Start to End	82.20	14.90	0.20	0.25	1.10	0.60	0.75	2.45	100%
m Road	Start to End	30.63	30.80	•	0.35	24.63	5.08	8.51	38.22	100%
Ibn Al Walid	Start to End	79.76	18.23	0.40	0.94	0.27	0.27	0.13	0.67	100%
ar Road	Start to End	73.94	22.11	0.95	2.68	0.32	•	•	0.32	100%
ubi Road	Start to End	76.76	21.52	•	0.92	0.56	0.15	•	0.71	100%
Matar Al Qadim	East Airport Rd.	67.60	27.50	•	1.43	2.00	1.16	0.31	3.47	100%
sity	Start to End	63.70	28.90	1.05	1.15	4.00	0.30	0.90	5.20	100%
h	Ramp, Makkah Ex to Air.	54.44	22.55	1.72	0.13	14.90	4.41	1.86	21.23	100%
h	North to Airport	72.70	18.65	•	•	7.30	0.70	0.65	8.65	100%
h	Airport to Tahlia	74.00	19.81	1.09	0.16	4.15	0.50	0.29	4.94	100%
h	South to Tahlia	75.53	18.65	1.20	1.42	1.96	0.24	•	2.20	100%
halid Road	South to Makkah Rd.	63.71	28.07	1.26	2.41	3.61	0.72	0.22	4.55	100%
halid Road	North to Makkah Rd.	77.26	19.44	0.65	1.54	0.93	0.28	•	1.21	100%
	Start to End	60.95	22.52	0.62	0.26	10.84	3.73	1.08	15.65	100%
Expressway		49.96	30.84	2.58	0.19	10.47	4.05	2.01	16.53	100%
Road	Outer to Inner King Road	66.69	26.18	1.87	1.36	3.90	•	•	3.90	100%
Road	King Khalid to Al Fahd	64.48	30.90	2.05	1.46	0.75	0.36	•	1.11	100%
Road	Al Fahd to Makkah Expressway	45.39	39.37	1.13	0.21	8.92	2.76	2.22	13.90	100%

Table 5.4.3-1 Observation of traffic composition at 32 locations from the city of Jeddah 1984.
(Reproduced from reference No.122)

(Reproduced from reference No. 122)

Road Section		Typical Week Day					Friday or Hajj Period				
	Code No.	Average Daily Volume	Capacity	V/C	Speed km/h	Level of Service	Average Daily Volume	Capacity	V/C	Speed km/h	Level of Service
Uninterrupted Flow											
Makkah Expressway	34	35000	210000	0.17	110	A	18300	210000	0.09	110	A
Madinah Road	29	19100	158000	0.11	90	B	15400	158000	0.08	100	A
Madinah Road	4	55200	203000	0.27	93	A	49600	203000	0.24	100	A
Airport Access Road	31	9000	110000	0.06	95	A	8000	110000	0.07	95	A
Interrupted Flow											
Makkah Road	1	57500	119000	0.48	26	D	35100	119000	0.29	36	C
Makkah Road	16	77400	87000	0.89	39	D	50600	87000	0.53	53	B
King Khalid Street	2	68600	119000	0.58	31	D	49400	119000	0.41	43	B
King Khalid Street	3	62500	108000	0.58	30	D	45000	108000	0.42	42	B
Al Tahliyyah Road	5	32600	119000	0.27	45	B	28200	119000	0.24	51	A
Prince Majed Bin	6	32000	144000	0.22	32	D	28200	144000	0.20	36	C
Makarona Street	7	22700	119000	0.19	33	C	18200	119000	0.15	37	C
Prince Fahd Street	8	42000	119000	0.35	28	D	28200	119000	0.24	49	A
Prince Fahd Street	12	63000	119000	0.53	30	D	37000	119000	0.31	50	A
Palestine Street	9	28000	108000	0.26	37	C	31600	108000	0.29	28	D
Palestine Street	10	54200	119000	0.46	29	D	43700	119000	0.37	49	A
Bani Malek Street	11	25200	108000	0.24	49	A	17700	108000	0.16	54	A
Al Zubaidi Street	13	19950	119000	0.17	32	D	13300	119000	0.11	45	A
Al Zubaidi Street	27	12350	100000	0.12	37	C	8550	100000	0.09	42	B
Hail Street	14	40300	115000	0.34	46	B	41200	119000	0.35	46	B
Hail Street	33	16200	100000	0.16	37	C	16900	100000	0.17	37	C
Al Andalus	15	45400	150000	0.30	82	A	59500	150000	0.40	52	A
Madinah Road	16	47300	100000	0.47	28	D	35500	100000	0.36	51	A
Madinah Road	26	95000	119000	0.80	21	E	53200	115000	0.22	30	D
Madinah Road	32	36000	115000	0.30	16	E	20000	119000	0.17	28	D
Khalid Ibn Al Walid	17	22000	87000	0.25	29	D	13400	87000	0.15	43	B
Al Jamabi-Al Matar	19	56500	108000	0.52	40	B	37400	108000	0.35	44	B
[illegible] Al Matar	20	36000	119000	0.32	77	A	27200	119000	0.23	84	A
King Abd Al Aziz	21	43800	43200	1.00	20	F	20400	43200	0.47	45	B
King Faysal Street	22	33200	108000	0.30	36	C	17600	108000	0.16	55	A
King Faysal Street	28	19000	108000	0.18	23	F	14300	108000	0.13	25	D
Al Matar Road	23	46000	65000	0.74	23	F	28100	65000	0.43	32	D
Mohamed Nassef	24	13700	80000	0.17	52	A	10000	80000	0.13	58	A
Qattubah Street	25	16400	80000	0.23	39	C	13800	80000	0.17	42	B
Al Sanafah	30	24900	65000	0.38	25	D	16600	65000	0.25	35	C
Stadium Street	35	16500	108000	0.15	20	D	8300	108000	0.08	30	C
Stadium Street	36	37000	115000	0.31	20	D	18500	119000	0.16	32	C
Stadium Street	37	14000	100000	0.13	40	C	7000	108000	0.07	46	B
[illegible] Gate 3 to Gate 4	38	15400	86500	0.17	40	C	12300	86500	0.14	47	B
[illegible] Street	39	31600	119000	0.27	30	C	23800	119000	0.20	45	B
Al Jarubi Street	40	16000	102000	0.16	40	C	8000	102000	0.08	50	A
Al Manjar	41	24700	119000	0.21	45	B	16600	119000	0.14	60	A
Al Zahid	42	35600	108000	0.33	31	D	24400	108000	0.23	47	B
Al Zaizal	43	26800	119000	0.23	52	A	17400	119000	0.15	60	A
Al [illegible]	44	37600	119000	0.32	32	C	24400	119000	0.20	47	D
Bin Laden	45	22000	119000	0.18	25	D	13300	119000	0.11	43	C
University	46	22000	119000	0.18	30	D	16500	119000	0.14	42	B
Al [illegible]	47	18000	108000	0.16	33	C	9500	108000	0.09	45	B

Table 5.4.3-2 Average daily traffic volumes during weekday and Friday Hajj period based on 18 hours counts at 47 locations from the city of Jeddah, 1982.

(Reproduced from reference No.122)

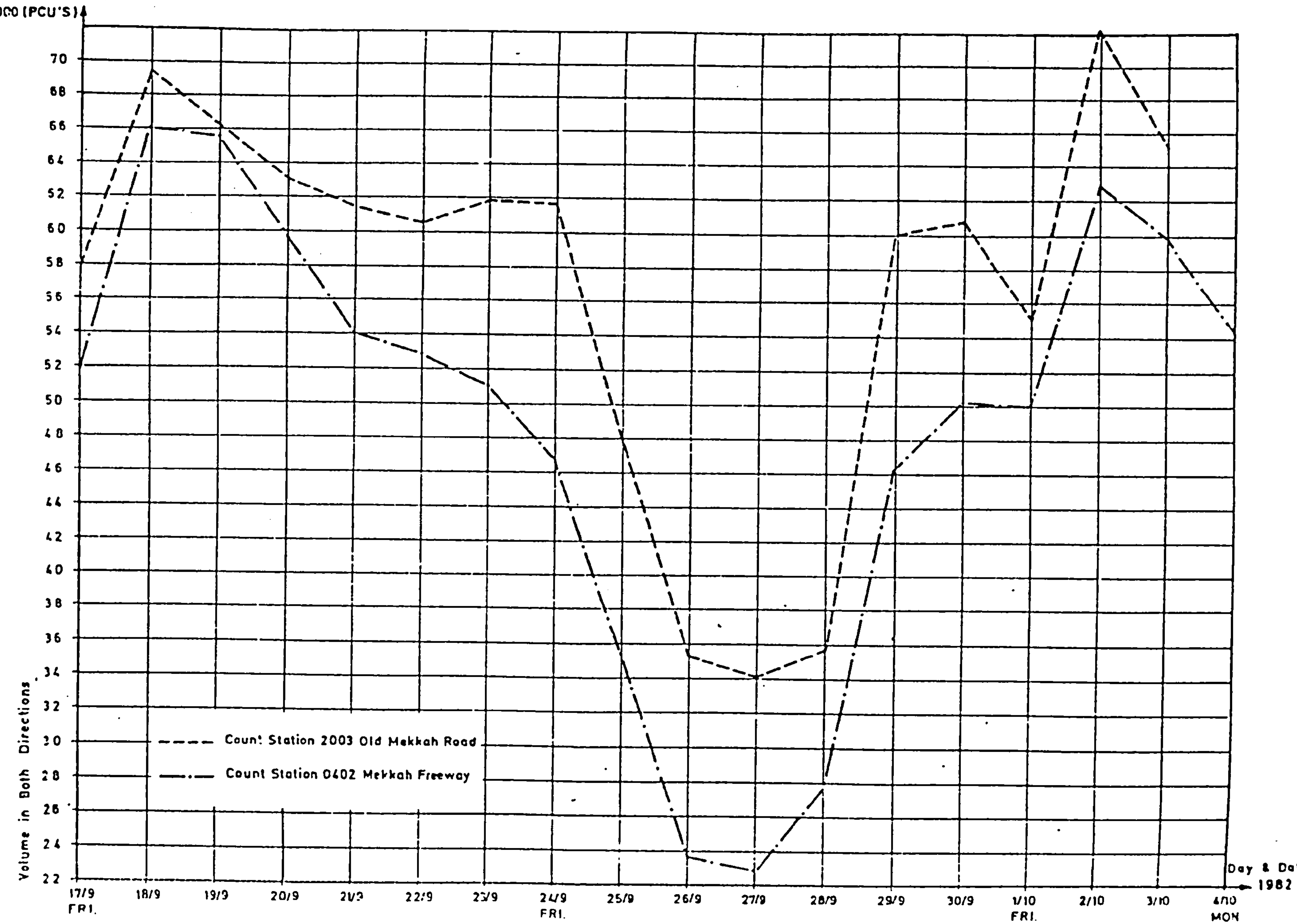


Figure 5. 4.3-1 Traffic volume variation during Hajj period (1982).

Many sections of the urban road network in Saudi Arabia experience heavy traffic volumes leading to congestion and long periods of delay when the network capacity is exceeded.

As an example, observations of traffic volumes at 47 locations in the city of Jeddah were carried out during 1952, using electronic counting and speed measuring equipment (122). The aim of these observations was to investigate the flow pattern of the city network by obtaining average daily flows based on an 18-hour count (6 am to midnight) and the traffic demand when sudden or periodical changes occur (Ramadan, Hajj period).

Traffic composition was investigated at 32 locations during 1984 and the results are given in table (5.5.3-1). Vehicle types were classified as passenger cars, pick-ups and mini buses (equivalent to light goods vehicles in the U.K.), motor cycles and trucks. Table (5.5.3-1) illustrates traffic composition on the road network of the city of Jeddah. Percentages of heavy commercial vehicles were recorded ranging from averages of 10 to 30 per cent whilst percentages of passenger cars ranged from 40 to 70 per cent. Other types of vehicles including buses, motor cycles and light goods vehicles ranged on average from 20 to 50 per cent.

The high percentages of heavy vehicles become even greater during the three months annual Hajj period. During this period, approximately two million people arrive at the city of Jeddah and start their pilgrimage to both Makka and Al-Madinah. The distance between Jeddah and Makka is 75 kilometres and Al-Madinah is 445 kilometres.

Observations of traffic volumes and speed in the city of Jeddah are given in table (5.5.3-2) for typical week days and during Friday in the Hajj period (Friday is the weekly holiday). The level of service for each road section is given for both a typical weekday and a Friday in the Hajj period. As can be seen, a distinction was made between uninterrupted and interrupted flows since different criteria were used to evaluate these sections. The volume and capacity ratio is also given and it can be noted that the v/c ratio for 81 per cent of road sections is less than 0.50, indicating that there is ample reserve capacity in the road network(122).Exceptions are the following sections:

- King Abdul Aziz Road (v/c = 1.00)
- King Faysal Road (v/c = 0.74)
- Old Makka Road (v/c = 0.89),

where the situation has become critical. During the 1982 Hajj period, observations were conducted at two counting stations, Makka Freeway and Old Makka Road and it was noted that the traffic composition was mainly buses and mini-buses. Traffic volume counts in Old Makka Road for fifteen continuous days showed an increase in traffic volume demand (approximately 64,000 p.c.u. daily) a week before Hajj and one week after, while a considerable decline (approximately 22,000 p.c.u. daily) is apparent in the middle of the Hajj period (see figure 5.5.3-1). Similar characteristics are shown for the Makka Freeway. Due to the high percentages of buses (ranging between 60 to 80 per cent of the total flow), the freeway was congested during most of the day-time hours when the level of service was estimated to vary between level E and level F.

5.5 - Traffic standards and specifications

5.5.1 - Traffic signals

In Saudi Arabia the majority of signals operate on a single fixed time plan throughout the day; this timing plan is developed generally without calculation and based primarily on observation of peak period traffic conditions. Lengthy stage times are adopted to increase capacity and the resultant cycle times may be close to, or in excess of, desirable maxima. Whilst this may be necessary to cater for peak period volumes, at other times of the day significant delays will be incurred and drivers become frustrated, causing them to disobey the signal indications. In the city of Jeddah there are 476 signalized intersections. They constitute one of the most important means of traffic management in the city and their operation is considered critical for handling the traffic. At present the operations are not satisfactory. The traffic signals in Jeddah are on high or low posts, the high ones usually hanging overhead. Depending on their location at four or three-way intersections and on intersections with pedestrians they are four, three and two-phase signals respectively. The cycle time of the phases recorded varies between 52 and 255 seconds. This compares to maximum international standard of 120 second cycles which is violated in about 37 traffic signals, most of which are located in critical intersections causing long queues of congested traffic with the predictable negative effects on the traffic volumes and their speeds (122). The reasons for such long cycle times are due to the complexity of traffic composition, their unpredictable distributions; each movement at the intersection is served independently and very rarely are two parallel traffic movements allowed to operate concurrently. This is a major shortcoming of the current traffic signalisation in Jeddah. A similar situation exists throughout the Kingdom's traffic signal system.

The presence of the traffic police at signalized and unsignalized intersections throughout the day usually facilitates the proper circulation and turning movements of vehicles in certain important areas, and when traffic signals are not functioning due to failure.

The sequence of signals in Saudi Arabia are similar to the American system which is "red, green, amber, red" and present legislation permits right turns on red (left turn in U.K. practice) at most of the intersections. In cases where heavy right-turns and cross-through traffic occur, a separate right arrow is installed.

The location of signal posts at the intersections seems to be generally satisfactory. A major problem with the location of the signals at the intersections is the fact that signals are placed before and after the intersection. This encourages drivers to advance towards the latter signal, stopping on red, right on the pedestrian crossing and often obstruction crossing pedestrians. Traffic police tried to solve such problems by imposing heavy penalties but this was unsuccessful. This common practice by drivers is mainly due to the presence of uneducated drivers, the lack of knowledge of traffic regulations and rules and the high percentage of foreign drivers who are unqualified and are employed by foreign based companies. The majority of such drivers are from the Far East.

The flow of traffic using a junction is unlikely to be constant throughout the day and thus signal capacity and timings should be calculated for different periods of the day. The pattern of traffic flow can be expected to be similar from one working day to the next and thus signal arrangements will usually also be similar. However, differences in flow between weekdays, weekends and public holidays may need separate capacity assessments. Similarly, seasonal variation for example, the effects of Ramadan, Hajj and school holidays also require separate calculations.

5.5.2 - Signal controlled junction design

The objective of any traffic signal installation is to obtain the most efficient use of available road space consistent with the maximum safety for the different types of vehicles and pedestrians using the junction. Frequently signals are installed at intersections where available road space is restricted and the opportunity for road widening is limited, and as a result layouts have to be used which are not necessarily ideal. In Saudi Arabia, road widths and road reserves are generally sufficient to permit intersection layouts with high design standards to be introduced.

Traffic volumes presently experienced in the Kingdom, in conjunction with the development of road network, should not in general create severe congestion. The road network except in the older city centres, has adequate capacity to handle both present day flows and if utilized efficiently, to cater for growth in the foreseeable future. The emphasis of signals operation in Saudi Arabia is therefore not necessarily on the enhancement of the capacity of the network, but on the regulation and control of traffic, and on the improvement of road safety. However, it will become increasingly important as traffic growth continues to ensure that maximum operational efficiency, consistent with safety, is achieved through suitable junction design.

Intersection design standards are based on both U.K. and U.S.A. practice, modified and adapted as required for Saudi Arabia. In particular larger kerb radii, turning circles and carriageway widths have been included for use in areas where large commercial vehicles regularly operate.

As part of the recommended improvements for signal phasing and operation, separate signalization of specific turning movements is proposed, where feasible, and where an exclusive turning movement is signalled separately, the traffic making the movement should ideally be physically segregated. This promotes good lane discipline, improves safety, eases enforcement, can increase capacity (by achieving better lane utilization), and permits the signal heads to be more directly related to the movement they control.

The main criteria considered in the design of signalized intersection are as follows:

(a) Drivers have sufficient advance warning of exactly which direction to take at junctions;

(b) Drivers are guided into the intended lane (or lanes) by road markings, supplemented if necessary, by appropriate signs; and

(c) Drivers have a clear view of the signals at the junction itself;

(d) The physical layout of the intersection should ensure that an appropriate number of traffic lanes are provided for each movement, in relation to the overall traffic demand for that movement;

(e) Wherever possible, the volume/capacity ratio should be approximately equal for all approaches and particularly for those approaches that are allocated to the same phase.

Close co-ordination between the design of the geometric intersection layout and the associated traffic signal installation is required at all stages. Alternative methods of intersection operation will require different lane and channelization configurations, and the capacity requirements of individual approaches may dictate the number of lanes to be provided. It is a common practice to install a primary and secondary signal, at least one primary associated with each stop-line and with each separately signalled movement. Two primaries are generally required for wide (three or more lane) approaches and for one-way streets. The primary signal is installed one metre beyond the stop-line and normally on the nearside (right) of the carriageway. Where two primary signals (duplicate primaries) are used, one is placed one metre beyond each end of the stop-line, see figure (5.5.2-1).

Additionally, on wide approaches and where traffic

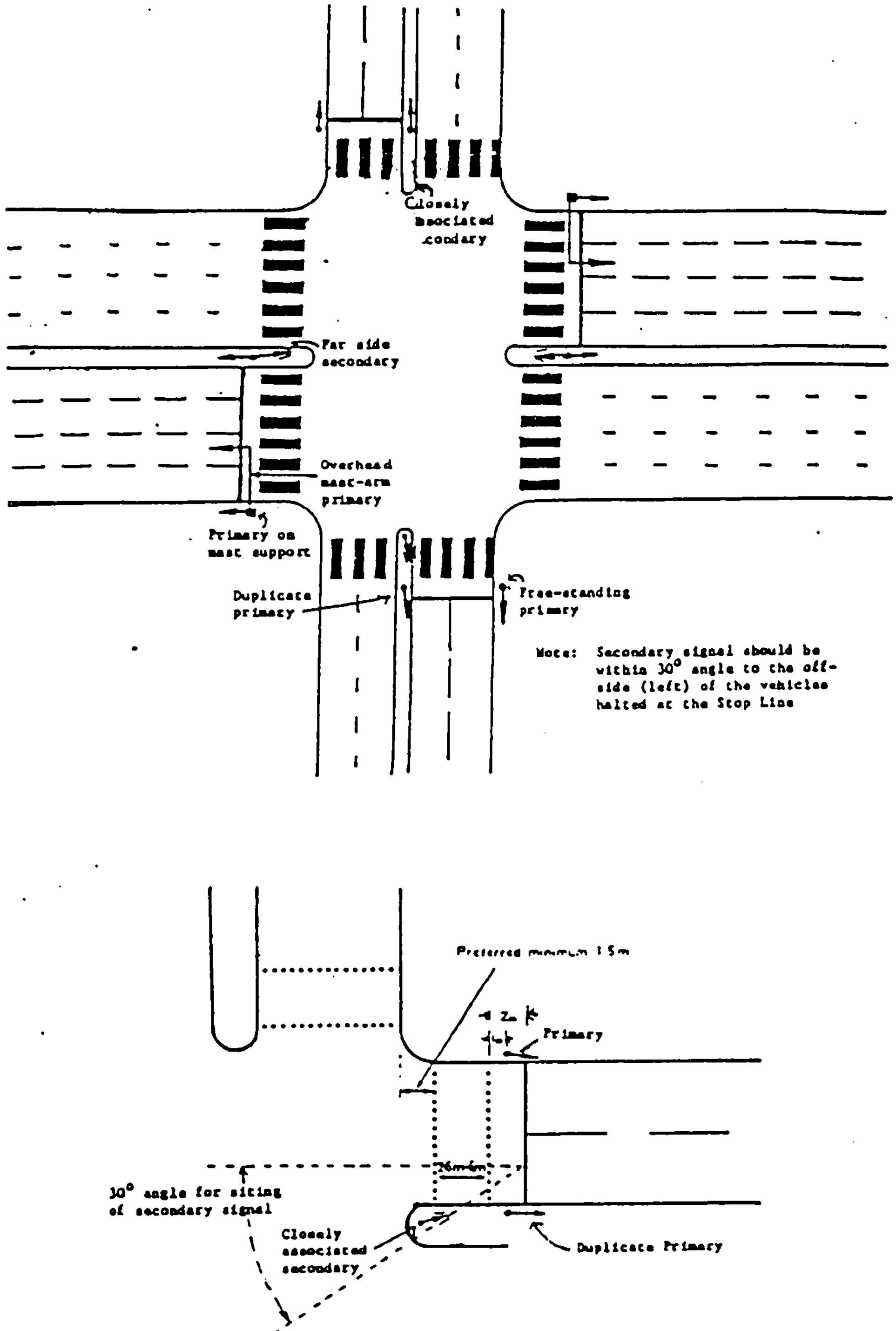


Figure 5.5.2-1 Typical signal pole positions (Saudi Arabia).
(Reproduced from reference No.124)

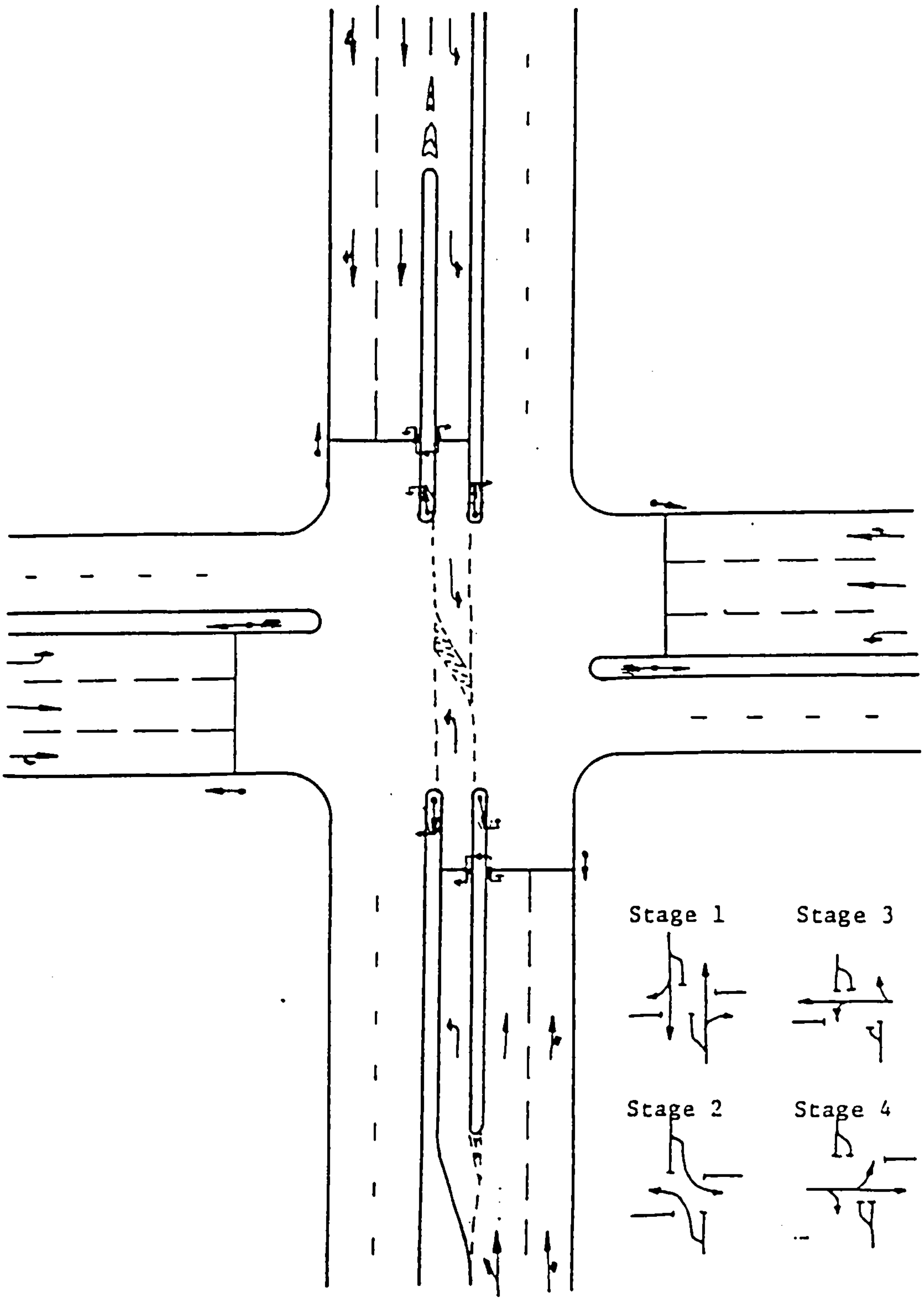
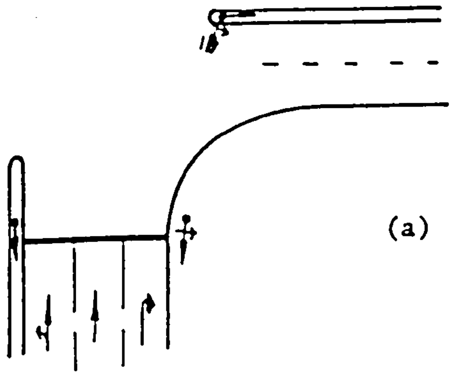
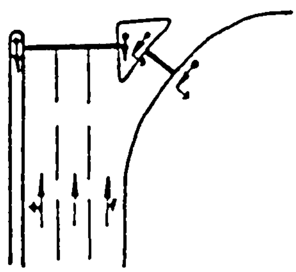


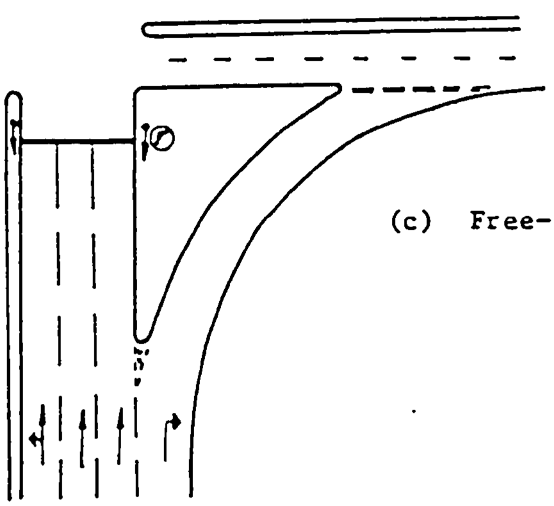
Figure 5.5.2-2 Signalized intersection with facilities for left turning traffic.
(Reproduced from reference No.124)



(a) Green Filter Arrow



(b) Separately Signalled Approach



(c) Free-flow Slip Road

Figure 5.5.2-3 Alternative facilities for right turning traffic.

(Reproduced from reference No.124)

speeds are high and the percentage of heavy commercial vehicles are significant, it is desirable to mount a third primary on an overhead mast arm. The mast for such overhead signals should generally be on the nearside of the carriageway one metre beyond the stop-line and the signal should control the main through-traffic only, see figure (5.5.2-1).

Examples of different designs of signalized intersections are shown by figures (5.5.2-2), (5.5.2-3) and (5.5.2-4). These figures show alternative solutions for different movements at the junction and junction widening. Typical layouts for right turn facilities (left turn in the U.K.) are shown by figure (5.5.2-3). Where a segregated facility cannot be provided, the right turn may be controlled by a green arrow, permitting the movement to take place in advance of the main green phase for the approach, figure (5.5.2-3 (a)). Left-turning facilities are as shown by figure (5.5.2-2) which also shows signal phasing. This layout demonstrates two alternative means of handling left turning traffic. In both cases, the median has been off-set to the left on each approach in order to maximise the available stop-line width; left-turning traffic, when halted at the stop-line , is directly in line with the opposing left-turning lane. This arrangement creates additional capacity at the stop-line whilst retaining on each exit sufficient capacity for the straight-ahead flow.

5.6 - Passenger car unit values at signalized intersections

In order to allow for the effects of vehicle type on junction capacity assessments, traffic composition is expressed in terms of passenger car units (PCU). . A factor is applied to each type of vehicle so that it is represented as an equivalent number of private cars. Capacities of junctions are usually expressed in PCU.

In Saudi Arabia, where a large variety of different vehicle types operate in urban areas, an appropriate simplified system is adopted based on the United Kingdom method and on research conducted at the University of Petroleum and Minerals (Dhahran), 1985. The proportion of vehicle types in the traffic stream varies throughout the day with greater changes during the peak hours. In the study area, buses constitute a smaller percentage and heavy commercial vehicles a higher percentage of the traffic. Pick-ups and mini buses account for an average of 23%, buses for 1.9%, heavy commercial vehicles 23% and motor cycles for 1.2%. Equivalent values of saturation flow in pcu's are included in table 5.6-2. Wherever data on traffic flow composition are available, then saturation flows are quoted in PCU, and capacity calculations undertaken accordingly.

Vehicle	Equivalence factor
Passenger car	1.00
Pick up and mini bus	1.25
Motor cycles	0.50
Bus	2.50
2 axle truck	2.00
3 axle truck	2.50
4 axle truck	3.50

Table 5.6-1 PCU factors for traffic signal design in Saudi Arabia.

(Reproduced from reference No.124)

Characteristics of Approach	Vehicles per hour per metre width of stop line	PCU* per hour per metre width of stop line
A Well aligned, wide unobstructed approach <u>and</u> exit; little or no interference from turning traffic	500	525
B Good visibility, wide unobstructed approach and exit; significant volume of unopposed turning traffic	475	500
C Speed of vehicles affected by alignment on entry or exit; some interference from turning traffic effects through movement	450	475
D Flow affected either by high proportion of turning traffic <u>or</u> loss of width due to obstructions on entry or exit, or free flow turns	425	450
E Interference to traffic from turning vehicles, incl. U-turns; parked vehicles impede traffic; shared use of lanes by turning traffic	400	425

*The above figures are for traffic composition of 20% heavy vehicles.

Table 5.6-2 Saturation flow rates per metre width of traffic signal approach.
(Reproduced from reference No.124)

CARRIAGEWAY WIDENING IN JUNCTIONS

Inside Corner Radius (m)	Single Lane Width (m)	Two Lane Width (m) Total	Inside Lane	Outside Lane
10	8.4	14.9	8.4	6.5
15	7.1	13.1	7.1	6.0
20	6.2	11.8	6.2	5.6
25	5.7	10.9	5.7	5.2
30	5.3	10.3	5.3	5.0
40	4.7	9.3	4.7	4.6
50	4.4	8.7	4.4	4.3
75	4.0	8.0	4.0	4.0
100	3.8	7.6	3.8	3.8
150	3.65	7.3	3.65	3.65

Note: In areas where long vehicles (in excess of 15.5m) operate, these values will need to be increased.

Table 5.6-3 Design values for traffic signal approach widths.
(Reproduced from reference No.124)

On-street parking or any other similar obstruction in the vicinity of a junction will restrict the capacity of the approach. To allow for the effect of parked vehicles at, or near a junction, the following procedure is used:

- if there is on-street parking within 7.5 metres of the stop-line, the effective stop-line width is reduced by 1.7 metres; or
- if there is on-street parking at a distance greater than 7.5 metres, the effective stop-line width is reduced by:

$$1.7 - 0.9\ (Z - 7.5)/g \quad \text{metres}$$

where g is the green time in seconds and Z is the distance of the nearest parked vehicle from the stop-line; the reduction is only made if the value of $0.9\ (Z - 7.5)/g$ is positive.

A number of experiments have been conducted to establish saturation flows in Saudi Arabian conditions ( ). A methodology based on U.K. and U.S.A. practice has been developed to relate saturation flow to the total vehicular flow on any approach under a number of different configurations, traffic mixes and proportions of turning traffic. Within the Kingdom, approach widths (number of lanes) are frequently relatively large and lane discipline is generally poor. This results in more vehicles at the stop-line than the number of marked lanes: stop-line width, rather than the number of lanes, is therefore a more reliable indicator of the capacity of an approach. As a result of the study, table (5.6-2) shows observed saturation flows for application in different circumstances.

The figures quoted in table (5.6-2), which are significantly lower than equivalent values of saturation flow adopted in the U.K. or the U.S.A., reflect local conditions, traffic composition and driver behaviour. It is interesting to note, that despite an apparent efficient use of road space by vehicles crowding the stop line, the saturation flow rate falls off rapidly after the initial discharge, as vehicles form a more orderly queue. The saturation flow is also affected by vehicles making turning manoeuvres from an incorrect lane, thereby impeding the rate of discharge.

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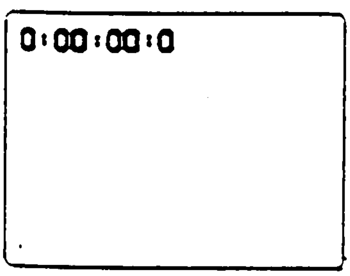
APPENDICES

Appendix A

Description of equipment used

Video Camera

The colour video camera (GX-N70E JVC) was used throughout this research study. The camera is equipped with remote control facility and quick-response infra-red auto-focus system. The camera has a 2-speed, 8X power zoom lens for smooth zooming from wide-angle to telephoto and vice versa which can be operated automatically or manually. An important feature of this colour camera is the eight page title facility and the date and elapsed time which can be super-imposed over the scene being recorded using three colours;red, blue or green. The elapsed time is displayed at the top left of the screen with an accuracy of 1/10th of a second as shown below:-



Stopwatch display area

- The elapsed time is displayed at the top left of the screen.
- It is displayed as it is displayed in the form: hours: minutes: seconds: 1/10ths of seconds (Example: 9:59:59:9)

  □:□□:□□:□
- 9 hours, 59 minutes 59.9 seconds is the maximum time that can be displayed.

The colour video camera lenses are powerful and newly developed with ulta-high sensitivity "Newvicon" which allows recording with a minimum illumination of 10 lux (1 footcandle). The camera is also equipped with uni-directional electret condenser microphone which has high sound sensitivity. The electronic viewfinder incorporates a 1" black and white CRT. The scene being shot can be monitored to determine correct exposure, focus, composition, battery powercheck and elapsed time. It can also be used to monitor playback.

Portable Colour Video Cassette Recorder

The (HR-2200EG/EK JVC) colour video cassette recorder was used with the afore mentioned colour video camera. The recorder is extremely compact and light-weight, it is easy to use and built to the VHS format. It has a flexible three power supply arrangement, exclusive Ni-Cd rechargeable battery pack, (household AC and car battery). The HR-2200EG/EK employs VHS type cassettes only; E-180 for 3 hours, E-120 for 2 hours, E-60 for 1 hour and E-30 for 30 minutes of recording. The recorder has full 12-mode remote control operation and feather-light touch push-button operation due to microprocessor-based full-logic tape control. Also the recorder is equipped with newly-developed slow motion mechanism for producing clean slow motion pictures, shuttle search for quick picture search in both directions at about 10 times the normal speed, and audio dubbing facility provided to record sound track on pre-recorded tapes. The recorder may be connected to most TV receivers due to the built-in RF converter. The recorder has a tape running indicator for easy visual check of the tape running condition and LCD (Liquid Crystal Display) tape counter with memory function.

Tuner/Adaptor

The tuner/adaptor (TU-24EK) serves as an AC power adaptor for the HR-2200EK portable recorder and quick-charging function for the NB-P1 rechargeable battery pack installed in the HR-2200EK. The TU-24EK is equipped with clock/timer memory-hold system with a built-in rechargeable battery facility and 12-channel electronic TV tuner, having push-button (feather-light touch) channel selectors.

AC Power Adaptor

The AC power adaptor (AA-P22EG/EK JVC) is used exclusively with the HR-2200 EG/EK portable video cassette recorder from an AC household outlet. The AA-P22EG/EK is able to recharge two NB-P1 battery packs in series, first the one installed in the recorder and then automatically the other installed in this adaptor.

Colour Monitor

The VM-20 PSN(G) colour monitor was used to display recorded tapes. The 20-inch monitor screen allows clear pictures during film analysis in the laboratory. The monitor is equipped with decoders for PAL, SECAM, NTSC 3.58 MH2 and NTSC 4.43 MH2. Also it has automatic switching between colour systems with manual over-ride and a panel control to adjust brightness, vertical hold, colour control, colour contrast and NTSC tint control.

Appendix B

Sample of input data

THORNTON ROAD / WHETLEY LANE TRAFFIC SIGNAL
((BRADFORD))

MAIN DATA FILE

SERIAL NO.	GREEN TIME	X1	X2	X3	X4	X5	X6	X7
1	24.26	10	2	2	2	2	1	2
2	24.70	9	3	2	2	1	2	1
3	22.32	12	1	2	1	1	1	2
4	23.48	12	3	0	2	1	2	1
5	21.94	6	3	2	2	2	2	2
6	24.47	9	1	2	3	1	2	2
7	21.25	12	2	2	0	1	1	2
8	25.20	10	0	3	2	1	3	2
9	22.00	8	2	2	2	2	0	3
10	23.80	9	3	2	2	2	3	0
11	21.56	9	2	2	1	0	2	2
12	22.54	10	1	2	2	1	2	1
13	26.07	9	3	2	3	2	2	1
14	24.34	12	1	2	2	1	1	2
15	24.24	13	2	2	2	1	1	0
16	23.88	12	1	2	2	2	0	2
17	25.19	14	1	2	1	0	2	2
18	23.44	12	3	0	2	1	1	2
19	21.46	11	2	2	0	1	2	2
20	24.94	12	2	2	2	0	2	1
21	27.34	13	1	2	3	2	1	2
22	25.41	10	4	3	2	1	0	1
23	25.30	11	1	4	2	1	2	0
24	30.10	16	3	2	2	1	2	1
25	30.43	16	2	2	2	0	3	2
26	26.99	12	2	1	3	2	3	1
27	24.94	10	0	2	3	1	3	1
28	27.40	14	2	2	3	2	0	1
29	25.44	8	5	0	3	1	3	2
30	22.08	10	4	3	0	2	2	0
31	27.31	12	3	3	2	1	0	2
32	27.16	13	2	1	3	2	1	2
33	23.55	11	2	2	1	1	2	2
34	30.10	16	2	2	2	0	3	1
35	25.48	12	1	3	2	1	0	2
36	29.05	14	1	4	2	1	1	1
37	25.96	10	0	0	4	2	0	2
38	31.42	14	2	3	2	1	3	2
39	25.08	9	4	2	3	0	1	1
40	24.93	10	3	2	1	1	3	2
41	23.92	13	3	1	1	1	2	1
42	25.59	11	4	2	2	1	2	0
43	23.54	14	2	3	1	2	2	2
44	24.53	12	1	3	2	1	1	1
45	21.40	10	2	4	0	2	2	0
46	27.49	12	0	3	4	1	0	2
47	25.60	12	3	2	2	1	2	0
48	22.45	11	1	3	1	2	0	2
49	23.20	14	2	2	3	0	1	1

50	23.24	10	4	2	0	2	2	2
51	24.37	13	3	0	2	1	1	2
52	23.30	15	0	1	3	2	2	2
53	25.01	16	2	1	1	2	2	1
54	21.33	10	3	2	1	1	1	1
55	25.40	14	1	2	2	1	1	1
56	26.80	13	2	1	3	2	2	1
57	24.48	11	4	2	1	1	2	1
58	25.32	11	2	3	3	1	1	1
59	24.04	11	0	1	3	2	2	2
60	25.49	14	1	0	3	2	1	2
61	23.27	13	3	2	0	1	2	1
62	25.90	15	1	2	2	0	1	1
63	23.05	12	3	2	1	1	0	2
64	24.76	12	2	2	2	2	2	0
65	25.83	12	2	3	2	1	1	1
66	25.08	10	2	3	3	1	1	1
67	24.31	14	1	1	2	1	2	1
68	23.55	13	2	2	1	2	1	1
69	23.25	12	2	3	2	1	2	2
70	23.48	13	2	1	2	2	1	1
71	24.91	8	3	2	2	3	3	1
72	24.73	13	0	3	1	1	1	3
73	24.75	14	2	1	2	2	1	1
74	24.52	12	2	1	2	1	1	2
75	25.40	11	2	2	2	0	2	2
76	27.50	13	1	2	3	2	1	2
77	26.04	9	3	2	3	2	2	1
78	27.40	14	2	2	3	2	0	1
79	31.50	16	2	2	2	0	3	2
80	30.90	16	3	2	2	1	2	1
81	31.40	14	2	3	2	3	2	2
82	27.27	15	1	1	2	1	2	2
83	29.38	12	3	2	3	1	3	1
84	24.40	13	2	2	1	1	1	2
85	25.30	14	1	2	1	2	2	2
86	26.36	15	2	2	1	1	2	1
87	27.63	16	2	1	2	1	1	2
88	28.38	15	1	3	2	1	1	1
89	25.43	14	2	2	1	1	2	1
90	25.80	12	3	2	2	2	1	1
91	27.84	13	1	2	3	2	1	2
92	24.34	12	1	2	2	1	1	2
93	25.99	12	2	1	3	2	3	1
94	24.94	10	0	2	3	1	3	1
95	24.93	10	3	2	1	1	3	2
96	25.60	12	3	2	2	1	2	0
97	25.01	16	2	1	1	2	2	1
98	25.40	14	1	2	2	1	1	1
99	25.83	12	2	3	2	1	1	1
100	24.81	14	1	1	2	1	2	1

20 / 7 / 1986

MANNINGHAM LANE / MARLBOROUGH RD. TRAFFIC SIGNLAL

** BRADFORD))

MAIN DATA FILE

SERIAL NO.	GREEN TIME	X1	X2	X3	X4	X5	X6	X7
1	22.52	8	3	1	2	2	3	1
2	20.84	10	1	2	1	2	1	2
3	21.42	9	2	1	2	3	2	1
4	20.19	9	0	2	1	2	2	2
5	19.79	11	1	0	2	1	1	2
6	26.07	9	3	2	3	2	2	1
7	24.34	12	1	2	2	1	1	2
8	24.24	13	2	2	2	1	1	0
9	23.88	12	1	2	2	2	0	2
10	25.19	14	1	2	1	0	2	2
11	23.44	12	3	0	2	1	1	2
12	21.46	11	2	2	0	1	2	2
13	24.94	12	2	2	2	0	2	1
14	27.84	13	1	2	3	2	1	2
15	30.10	16	3	2	2	1	2	1
16	30.43	16	2	2	2	0	3	2
17	26.99	12	2	1	3	2	3	1
18	24.94	10	0	2	3	1	3	1
19	27.40	14	2	2	3	2	0	1
20	25.44	8	5	0	3	1	3	2
21	27.81	12	3	3	2	1	0	2
22	27.16	13	2	1	3	2	1	2
23	30.10	16	2	2	2	0	3	1
24	29.05	14	1	4	2	1	1	1
25	31.42	14	2	3	2	1	3	2
26	24.93	10	3	2	1	1	3	2
27	25.59	11	4	2	2	1	2	0
28	28.54	14	2	3	1	2	2	2
29	24.53	12	1	3	2	1	1	1
30	25.60	12	3	2	2	1	2	0
31	23.24	10	4	2	0	2	2	2
32	24.37	13	3	0	2	1	1	2
33	28.36	15	0	1	3	2	2	2
34	25.01	16	2	1	1	2	2	1
35	21.33	10	3	2	1	1	1	1
36	25.40	14	1	2	2	1	1	1
37	26.88	13	2	1	3	2	2	1
38	24.48	11	4	2	1	1	2	1
39	25.82	11	2	3	3	1	1	1
40	23.27	13	3	2	0	1	2	1
41	25.90	15	1	2	2	0	1	1
42	29.55	14	2	2	3	1	2	1
43	26.71	16	1	1	2	1	2	1
44	26.70	15	2	2	1	2	2	1
45	25.15	13	3	1	2	1	1	1
46	25.05	15	2	1	1	1	2	2
47	23.65	12	3	2	1	1	0	2
48	24.76	12	2	2	2	2	2	0
49	30.90	16	3	2	2	1	2	1
50	29.38	12	3	2	3	1	3	1
51	30.59	13	3	3	2	1	2	2
52	26.20	10	2	2	2	2	3	2
53	30.59	13	3	3	2	1	2	2

*	54	29.55	14	2	2	3	1	2	1	*
*	55	30.59	13	3	3	2	1	2	2	*
*	56	22.94	12	1	2	1	2	2	1	*
*	57	29.55	14	2	2	3	1	2	1	*
*	58	30.59	13	3	3	2	1	2	2	*
*	59	20.30	7	2	1	2	2	2	2	*
*	60	30.59	13	3	3	2	1	2	2	*
*	61	29.55	14	2	2	3	1	2	1	*
*	62	30.59	13	3	3	2	1	2	2	*
*	63	26.05	15	2	1	1	1	2	2	*
*	64	30.10	16	2	2	2	2	1	2	*
*	65	33.90	14	2	4	1	2	3	2	*
*	66	30.59	13	3	3	2	1	2	2	*
*	67	30.10	16	2	2	2	2	1	2	*
*	68	33.90	14	2	4	1	2	3	2	*
*	69	24.75	13	2	2	1	1	2	1	*
*	70	31.90	14	2	4	1	2	3	2	*
*	71	30.30	16	2	2	2	2	1	2	*
*	72	30.59	13	3	3	2	1	2	2	*
*	73	29.15	15	4	1	1	2	2	2	*
*	74	32.13	15	2	3	2	2	3	1	*
*	75	31.90	14	2	4	1	2	3	2	*
*	76	30.30	16	2	2	2	2	1	2	*
*	77	31.72	14	2	4	1	2	3	2	*
*	78	24.42	13	2	2	1	1	2	1	*
*	79	32.13	15	2	3	2	2	3	1	*
*	80	31.90	14	2	4	1	2	3	2	*
*	81	32.13	15	2	3	2	2	3	1	*
*	82	30.30	16	2	2	2	2	1	2	*
*	83	30.59	13	3	3	2	1	2	2	*
*	84	31.70	13	3	3	2	3	1	3	*
*	85	29.15	15	4	1	1	2	2	2	*
*	86	30.50	16	2	2	2	2	1	2	*
*	87	32.40	14	2	4	1	2	3	2	*
*	88	29.14	15	4	1	1	2	2	2	*
*	89	25.57	10	2	3	1	1	4	1	*
*	90	32.45	16	1	3	2	1	3	2	*
*	91	29.55	14	2	2	3	1	2	1	*
*	92	30.59	13	3	3	2	1	2	2	*
*	93	30.10	16	2	2	2	2	1	2	*
*	94	33.90	14	2	4	1	2	3	2	*
*	95	31.90	14	2	4	1	2	3	2	*
*	96	29.15	15	4	1	1	2	2	2	*
*	97	32.13	15	2	3	2	2	3	1	*
*	98	31.72	14	2	4	1	2	3	2	*
*	99	31.70	13	3	3	2	3	1	3	*
*	100	32.10	14	2	4	1	2	3	2	*
*										*

20 / 7 / 1986

21.30.47.UCLP, C2, X71LP , 0.128KLNS.

WOODHOUSE LANE / HYDE PARK RD. TRAFFIC SIGNAL

((LEEDS))

MAIN DATA FILE

SERIAL NO.	GREEN TIME	X1	X2	X3	X4	X5	X6	X7
1	27.90	17	2	2	1	2	1	1
2	27.71	14	2	1	3	1	2	1
3	28.87	18	3	2	0	3	2	1
4	29.31	15	2	2	3	1	1	1
5	30.00	19	1	2	2	2	2	0
6	29.56	16	0	1	4	1	1	2
7	25.31	15	3	2	1	1	0	1
8	30.06	19	1	0	3	1	2	1
9	30.55	18	2	2	2	2	1	1
10	28.20	15	3	2	2	1	1	1
11	31.80	17	2	1	3	0	2	2
12	29.46	16	3	2	2	1	1	1
13	30.46	16	0	4	2	1	1	2
14	27.65	14	3	2	3	1	1	1
15	28.70	16	2	2	2	2	0	2
16	28.81	18	1	0	3	1	2	1
17	29.00	18	3	2	1	0	1	1
18	30.30	13	3	3	3	2	1	1
19	31.01	19	1	1	3	1	2	1
20	29.96	17	2	2	3	1	1	0
21	32.16	17	2	3	2	1	2	1
22	25.40	14	1	3	0	2	2	2
23	31.21	17	2	2	3	1	1	1
24	32.01	15	2	3	3	1	2	1
25	27.41	15	1	2	2	1	2	1
26	29.90	14	3	2	3	2	1	1
27	30.06	16	2	3	2	1	1	1
28	28.65	18	1	1	3	0	1	1
29	30.81	16	0	3	3	1	2	1
30	27.65	17	2	2	1	2	1	1
31	27.06	17	4	1	1	1	2	0
32	28.56	13	3	2	3	1	1	1
33	27.15	15	2	0	3	2	2	2
34	30.31	15	2	2	3	1	2	1
35	29.11	16	1	3	2	1	1	1
36	29.95	13	4	2	3	0	2	1
37	29.25	13	2	3	3	2	1	1
38	26.45	18	2	2	0	2	1	1
39	30.00	16	1	1	3	2	2	2
40	28.65	19	1	2	1	2	1	1
41	28.86	16	2	3	2	1	0	1
42	27.81	16	2	1	2	1	2	1
43	29.70	18	1	1	3	2	2	0
44	27.31	17	2	2	1	1	1	1
45	29.51	18	1	1	3	1	1	1
46	28.31	15	1	3	2	1	1	1
47	29.15	15	2	3	2	1	1	1
48	31.00	15	2	2	3	2	1	2
49	26.51	19	1	1	2	1	1	1
50	29.31	19	2	2	1	1	1	1
51	30.30	17	1	3	2	2	1	1
52	31.11	16	2	2	3	1	2	1

*	53	28.31	14	2	3	2	1	1	1	*
*	54	28.16	14	3	3	1	1	2	1	*
*	55	30.01	18	0	2	3	1	1	1	*
*	56	29.90	15	2	2	2	2	2	2	*
*	57	26.81	19	1	0	2	1	1	1	*
*	58	28.31	16	2	2	2	1	1	1	*
*	59	28.56	17	1	1	3	1	1	1	*
*	60	29.01	16	1	3	2	1	1	1	*
*	61	28.36	18	3	3	0	1	1	1	*
*	62	31.50	18	2	2	2	2	2	1	*
*	63	30.20	18	1	1	3	0	2	1	*
*	64	28.71	15	3	2	2	1	1	1	*
*	65	32.92	17	1	3	3	1	1	2	*
*	66	28.92	14	4	1	2	1	3	1	*
*	67	30.62	13	3	3	3	1	1	1	*
*	68	27.75	19	1	1	2	2	0	1	*
*	69	27.17	17	2	2	1	1	2	0	*
*	70	29.35	16	0	3	2	2	1	2	*
*	71	32.82	16	1	3	3	1	0	2	*
*	72	28.85	15	2	1	3	2	2	1	*
*	73	26.45	18	2	2	0	2	1	1	*
*	74	28.87	17	3	0	3	1	1	1	*
*	75	29.70	19	2	1	2	2	2	0	*
*	76	31.75	17	2	2	3	0	2	1	*
*	77	26.80	15	3	2	1	2	1	1	*
*	78	31.17	15	1	1	4	1	2	2	*
*	79	31.11	16	2	3	2	1	1	2	*
*	80	32.66	19	0	3	3	1	1	1	*
*	81	28.90	19	1	1	2	2	0	2	*
*	82	28.52	17	2	1	2	1	2	1	*
*	83	29.87	18	3	0	3	1	1	1	*
*	84	30.50	15	3	3	2	2	1	1	*
*	85	29.27	16	4	2	2	1	1	0	*
*	86	29.60	18	1	1	1	0	4	2	*
*	87	27.00	17	2	2	0	3	2	1	*
*	88	33.75	16	3	2	3	2	2	2	*
*	89	31.92	15	2	3	3	1	2	1	*
*	90	32.35	17	3	1	3	0	2	2	*
*	91	28.87	18	3	2	0	3	2	1	*
*	92	29.31	15	2	2	3	1	1	1	*
*	93	30.60	19	1	2	2	2	2	0	*
*	94	30.55	18	2	2	2	2	1	1	*
*	95	30.46	16	0	4	2	1	1	2	*
*	96	30.30	13	3	3	3	2	1	1	*
*	97	29.96	17	2	2	3	1	1	0	*
*	98	31.11	16	2	2	3	1	2	1	*
*	99	30.01	18	0	2	3	1	1	1	*
*	100	28.86	18	3	3	0	1	1	1	*
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22.01.56.UCLP, C2, X71LP , 0.123KLNS.

HEADINGLEY LANE / WOODHOUSE ST. TRAFFIC SIGNAL

((LEEDS))

MAIN DATA FILE

SERIAL NO.	GREEN TIME	X1	X2	X3	X4	X5	X6
1	29.15	18	0	2	3	1	1
2	27.64	15	2	2	2	2	2
3	25.85	19	1	0	2	1	1
4	27.25	16	2	2	2	1	1
5	27.35	17	1	1	3	1	1
6	27.80	16	1	3	2	1	1
7	27.80	18	3	3	0	1	1
8	27.50	13	3	2	3	1	1
9	26.50	15	2	0	3	2	2
10	29.50	15	2	2	3	1	2
11	27.87	16	1	3	2	1	1
12	29.00	13	4	2	3	0	2
13	28.65	13	2	3	3	2	1
14	25.45	18	2	2	0	2	1
15	27.14	16	0	3	2	2	1
16	28.60	16	1	3	3	1	0
17	27.84	15	2	1	3	2	2
18	25.44	18	2	2	0	2	1
19	27.78	17	3	0	3	1	1
20	29.88	19	2	1	2	2	2
21	30.80	17	2	2	3	0	2
22	29.64	18	2	2	2	2	1
23	27.20	15	3	2	2	1	1
24	29.40	17	2	1	3	0	2
25	28.40	16	3	2	2	1	1
26	28.07	16	0	4	2	1	1
27	28.08	14	3	2	3	1	1
28	26.24	16	2	2	2	2	0
29	27.70	18	1	0	3	1	2
30	30.29	15	3	3	2	2	1
31	29.47	16	4	2	2	1	1
32	27.40	18	1	1	1	0	4
33	25.50	17	2	2	0	3	2
34	31.56	16	3	2	3	2	2
35	30.87	15	2	3	3	1	2
36	30.20	17	3	1	3	0	2
37	30.87	15	2	3	3	1	2
38	26.28	15	1	2	2	1	2
39	28.84	14	3	2	3	2	1
40	28.83	16	2	3	2	1	1
41	27.90	18	1	1	3	0	1
42	29.68	16	0	3	3	1	2
43	26.50	17	2	2	1	2	1
44	27.88	17	4	1	1	1	2
45	25.76	15	3	2	1	2	1
46	28.77	15	1	1	4	1	2
47	28.82	16	2	3	2	1	1
48	31.41	19	0	3	3	1	1
49	26.54	19	1	1	2	2	0
50	27.47	17	2	1	2	1	2
51	28.78	18	3	0	3	1	1

52	26.76	17	2	2	1	2	1
53	26.48	14	2	1	3	1	2
54	28.04	18	3	2	0	3	2
55	28.08	15	2	2	3	1	1
56	30.64	19	1	2	2	2	2
57	27.18	16	0	1	4	1	1
58	24.28	15	3	2	1	1	0
59	28.98	19	1	0	3	1	2
60	28.02	15	2	3	2	1	1
61	28.72	15	2	2	3	2	1
62	27.28	19	1	1	2	1	1
63	28.16	19	2	2	1	1	1
64	29.07	17	1	3	2	2	1
65	30.07	16	2	2	3	1	2
66	27.17	14	2	3	2	1	1
67	27.17	14	3	3	1	1	2
68	27.74	16	1	1	3	2	2
69	27.44	19	1	2	1	2	1
70	27.72	16	2	3	2	1	0
71	26.67	16	2	1	2	1	2
72	29.72	18	1	1	3	2	2
73	26.18	17	2	2	1	1	1
74	28.26	18	1	1	3	1	1
75	27.07	15	1	3	2	1	1
76	30.40	18	2	2	2	2	2
77	29.00	18	1	1	3	0	2
78	27.56	15	3	2	2	1	1
79	28.87	17	1	3	3	1	1
80	27.77	14	4	1	2	1	3
81	29.47	13	3	3	3	1	1
82	26.50	19	1	1	2	2	0
83	27.17	17	2	2	1	1	2
84	27.75	18	3	2	1	0	1
85	28.90	13	3	3	3	2	1
86	30.32	19	1	1	3	1	2
87	29.96	17	2	2	3	1	1
88	30.91	17	2	3	2	1	2
89	26.05	16	2	3	0	2	2
90	29.96	17	2	2	3	1	1
91	29.00	13	4	2	3	0	2
92	29.88	19	2	1	2	2	2
93	30.29	15	3	3	2	2	1
94	31.56	16	3	2	3	2	2
95	30.87	15	2	3	3	1	2
96	28.83	16	2	3	2	1	1
97	28.77	15	1	1	4	1	2
98	28.08	15	2	2	3	1	1
99	28.72	15	2	2	3	2	1
100	30.07	16	2	2	3	1	2

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~~22.13.05.UGLP, G2, X71LP~~

STREATHAM HILL / STREATHAM PL. TRAFFIC SIGNAL

((LONDON))

MAIN DATA FILE

SERIAL NO.	GREEN TIME	X1	X2	X3	X4	X5
1	25.56	16	4	2	2	3
2	27.56	19	2	1	2	2
3	27.91	15	3	3	2	2
4	28.04	17	3	2	2	1
5	27.54	18	3	1	2	1
6	23.09	13	4	2	1	3
7	23.20	15	4	0	2	0
8	27.98	19	2	2	1	4
9	29.27	16	3	3	2	3
10	27.95	14	4	2	3	1
11	25.68	20	3	1	0	2
12	29.17	17	2	3	2	3
13	27.40	16	3	2	2	2
14	26.05	18	2	2	1	1
15	27.95	19	0	3	2	0
16	26.15	18	3	1	1	3
17	28.30	16	2	4	1	4
18	25.20	15	2	2	2	2
19	28.20	17	1	2	3	2
20	28.65	14	4	2	3	3
21	24.35	15	3	1	2	1
22	25.40	14	3	2	2	2
23	24.95	16	2	1	2	3
24	23.56	16	1	2	1	3
25	29.48	20	3	2	1	2
26	26.56	14	2	2	0	3
27	29.25	19	4	1	2	1
28	27.10	20	3	1	1	0
29	24.92	15	3	2	1	3
30	23.75	17	2	1	1	2
31	26.08	14	3	2	2	4
32	24.71	18	2	1	1	2
33	23.55	20	1	1	0	2
34	24.40	14	4	2	1	2
35	26.85	17	3	2	1	3
36	24.35	18	2	1	1	1
37	23.58	18	1	1	1	2
38	24.00	19	1	1	1	2
39	26.07	15	3	2	2	1
40	23.40	15	2	2	1	2
41	25.65	16	3	2	1	3
42	27.30	15	2	2	3	2
43	25.07	17	2	2	1	1
44	28.60	19	0	4	1	2
45	26.40	18	2	1	1	2
46	28.37	16	3	1	3	3
47	27.25	19	2	1	2	1
48	28.08	15	3	3	2	2
49	25.59	17	2	2	1	3
50	30.14	14	4	3	3	3

*	51	26.58	18	2	1	2	2	*
*	52	28.58	20	3	0	2	4	*
*	53	26.00	17	2	2	2	0	*
*	54	25.58	17	2	1	2	2	*
*	55	28.38	15	3	2	3	2	*
*	56	28.04	20	0	3	1	3	*
*	57	28.14	20	2	3	0	3	*
*	58	27.40	14	3	2	3	1	*
*	59	26.40	15	3	2	2	2	*
*	60	26.59	16	2	2	2	3	*
*	61	27.95	16	2	2	3	1	*
*	62	24.85	16	1	2	2	1	*
*	63	23.78	16	3	1	1	2	*
*	64	27.60	15	3	3	2	1	*
*	65	25.20	16	1	2	2	2	*
*	66	24.95	18	1	2	1	1	*
*	67	26.90	19	2	1	2	0	*
*	68	27.95	18	2	3	1	2	*
*	69	26.00	18	2	1	2	2	*
*	70	27.95	16	2	2	3	1	*
*	71	27.85	17	1	2	3	1	*
*	72	28.38	15	3	2	3	2	*
*	73	30.14	16	4	3	2	3	*
*	74	24.35	18	2	1	1	1	*
*	75	25.95	16	2	2	2	1	*
*	76	27.85	19	1	1	2	1	*
*	77	27.78	20	1	0	3	2	*
*	78	24.60	15	2	3	1	1	*
*	79	24.93	15	2	3	1	2	*
*	80	23.69	15	2	2	1	3	*
*	81	27.15	18	3	2	1	1	*
*	82	25.95	16	2	2	2	1	*
*	83	27.40	16	2	2	1	2	*
*	84	27.10	19	0	2	2	2	*
*	85	31.05	19	1	3	3	0	*
*	86	28.50	20	1	2	2	0	*
*	87	27.50	18	1	1	3	2	*
*	88	28.40	17	3	2	2	2	*
*	89	27.20	17	2	2	2	2	*
*	90	27.69	16	3	2	2	3	*
*	91	25.95	16	3	2	1	1	*
*	92	28.04	17	3	2	2	1	*
*	93	27.95	14	4	2	3	1	*
*	94	24.35	15	3	1	2	1	*
*	95	24.35	18	2	1	1	1	*
*	96	28.00	19	0	4	1	2	*
*	97	27.25	19	2	1	2	1	*
*	98	27.40	14	3	2	3	1	*
*	99	24.95	18	1	2	1	1	*
*	100	27.85	19	1	1	2	1	*
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22.24.41.UCLP, C2, X71LP , 0.128KLNS.

CHRISTCHURCH / STREATHAM HILL TRAFFIC SIGNAL

((LONDON))

MAIN DATA FILE

SERIAL NO.	GREEN TIME	X1	X2	X3	X4	X5	X6	X7
1	23.65	12	0	2	1	1	2	3
2	21.50	9	2	0	2	1	3	2
3	22.93	13	1	2	0	2	1	3
4	22.40	10	3	2	1	0	2	1
5	23.65	14	2	1	2	1	0	1
6	22.00	11	1	2	2	1	2	0
7	25.75	12	2	1	3	1	2	1
8	24.70	14	1	2	1	1	1	2
9	25.50	13	1	2	2	1	2	1
10	24.18	10	2	2	1	2	2	3
11	28.05	9	3	2	3	2	3	2
12	24.00	11	2	1	2	2	1	1
13	21.60	11	2	2	1	2	2	0
14	24.20	12	1	3	1	1	0	3
15	27.05	9	3	3	2	0	2	2
16	22.55	14	1	2	0	1	1	2
17	23.65	12	2	0	2	1	2	2
18	26.33	11	0	3	2	2	2	3
19	21.40	14	1	0	1	1	1	2
20	21.55	11	3	2	0	1	2	1
21	24.55	13	1	1	2	0	2	2
22	21.25	10	2	2	2	1	0	1
23	23.55	10	2	1	3	1	3	0
24	24.70	13	1	2	1	1	2	2
25	25.25	12	2	1	2	1	2	2
26	26.75	13	1	3	1	2	2	2
27	25.55	12	2	2	2	1	2	1
28	26.58	10	0	3	2	2	3	3
29	26.20	14	1	0	3	1	2	2
30	20.05	11	2	1	0	1	2	2
31	23.75	11	2	3	1	0	2	1
32	24.95	14	1	1	2	1	0	3
33	23.20	13	1	2	2	1	1	0
34	23.08	9	3	2	1	2	3	1
35	25.13	12	1	2	1	2	2	3
36	25.78	12	1	3	1	2	2	2
37	24.37	12	1	3	1	1	1	2
38	24.87	14	1	1	2	1	1	2
39	26.27	11	2	2	3	1	2	1
40	23.22	13	1	1	1	2	2	2
41	24.77	10	2	2	2	1	3	1
42	24.22	12	1	1	2	1	2	2
43	25.00	12	1	2	2	2	1	2
44	24.45	14	1	1	1	2	2	2
45	26.52	14	1	2	2	1	2	1
46	27.17	14	1	1	2	1	2	3
47	26.30	13	1	0	3	1	2	3
48	22.18	10	2	2	0	2	3	2
49	25.40	12	1	2	2	0	2	2
50	21.44	9	3	2	2	1	0	1
51	23.94	9	2	3	2	1	3	0

52	26.60	11	0	3	2	2	3	2
53	27.23	14	1	1	3	2	1	2
54	24.00	12	1	2	1	2	2	2
55	25.53	14	1	2	1	1	2	2
56	23.88	11	2	1	2	1	3	1
57	23.73	11	2	2	1	1	2	2
58	25.33	13	1	4	1	1	2	0
59	22.76	13	1	2	1	2	0	2
60	26.30	10	2	2	3	0	2	2
61	23.86	14	2	2	0	2	2	1
62	22.13	11	3	0	2	1	2	1
63	27.44	14	0	2	2	1	2	3
64	25.85	14	1	1	2	1	1	3
65	24.01	12	1	1	2	1	2	2
66	23.82	10	3	1	2	1	3	1
67	24.97	13	1	1	2	1	2	2
68	21.77	12	2	1	1	1	2	1
69	26.32	12	1	2	2	1	3	2
70	25.88	9	2	2	3	2	3	1
71	21.88	11	2	1	2	2	1	1
72	22.25	9	2	3	1	2	2	1
73	22.34	11	2	2	1	1	2	1
74	22.15	11	2	1	1	1	2	2
75	24.45	14	1	2	1	1	1	2
76	26.02	12	0	3	2	1	1	3
77	23.48	11	1	0	3	2	2	2
78	24.23	14	1	3	0	2	2	1
79	23.90	13	1	2	2	0	1	1
80	22.76	14	1	1	2	2	0	1
81	23.05	14	1	2	1	1	2	0
82	26.45	9	2	3	2	1	2	3
83	24.93	9	3	2	2	2	3	1
84	26.05	10	2	2	3	1	2	2
85	24.65	11	1	3	1	2	2	2
86	23.34	13	1	2	1	1	2	1
87	23.05	10	2	1	2	1	2	2
88	24.99	11	2	1	2	1	3	2
89	24.05	11	1	2	2	1	2	2
90	24.30	14	1	1	1	2	1	3
91	28.05	9	3	2	3	2	3	2
92	27.05	9	3	3	2	0	2	2
93	25.50	13	1	2	2	1	2	1
94	25.55	12	2	2	2	1	2	1
95	26.75	13	1	3	1	2	2	2
96	25.78	12	1	3	1	2	2	2
97	26.27	11	2	2	3	1	2	1
98	25.00	12	1	2	2	2	1	2
99	27.17	14	1	1	2	1	2	3
100	27.23	14	1	1	3	2	1	2

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~~22.37.39.UCLP, 62, X7+LP~~ ~~Q+28KLNG~~

Appendix C

Sample of computer output

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 I
 S
                                                                      86/12/20.    14.45.18.    PAGE   1
   COMPU*ER CEN*RE
   UNIVERSITY OF BRADFORD

   S P S S - - STATISTICAL PACKAGE FOR THE SOCIAL SCIENCES

   VERSION 8.3 (NOS) -- MAY 04, 1982

   376500 CM MAXIMUM FIELD LENG*H REQUES*


              RUN NAME         REGS1
              VARIABLE LIST    A,Y,X1 TO X7
              INPU* FORMA*     FREEFIELD
              N OF CASES       100

 CPU *IME REQUIRED..      .054 SECONDS


              REGRESSION       VARIABLES=Y,X1 *O X7
                               REGRESSION=Y WITH X1 TO X7
              OP*IONS          19
              STATISTICS       ALL
              READ INPUT DATA

        00053400 CM NEEDED FOR REGRESSION


        OPTION - 1
        IGNORE MISSING VALUE INDICA*ORS
        (NO MISSING VALUES DEFINED...OP*ION 1 MAY HAVE BEEN FORCED)

        OPTION -19
        FORCE REGRESSION *HRU *HE ORIGIN


1REGS1                                                                86/12/20..   14.45.18.    PAGE   2

 FILE    NONAME   (CREATION DATE =  86/12/20. )
0* * * * * * * * * * * * * * * * * * * * * * *  M U L * I P L E   R E G R E S S I O N  * * * * * * * * * * * * * * * * * * * * * * *
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X1	.99001						
X2	.87431	.84190					
X3	.91964	.82285	.79207				
X4	.92749	.90094	.75552	.80829			
X5	.88083	.86386	.77952	.79551	.82840		
X6	.88031	.85338	.81193	.73659	.79055	.75171	
X7	.89702	.88658	.73261	.80145	.82180	.79513	.74533
	Y	X1	X2	X3	X4	X5	X6

FILE NONAME (CREA*ION DA*E = 86/12/20.)

0* M U L T I P L E R E G R E S S I O N *

0DEPENDEN* VARIABLE.. Y —— REGRESSION FORCED THROUGH ORIGIN ——

MEAN RESPONSE 25.58160 S*D. DEV. 25.68535

VARIABLE(S) EN*ERED ON S*EP NUMBER 1.. X1
X2
X5
X6
X7
X3
X4

MUL*IPLE R	.99993	ANALYSIS OF VARIANCE	DF	SUM OF SQUARES	MEAN SQUARE	F SIGNIFICANC
R SQUARE	.99987	REGRESSION	7.	65967.44633	9423.92090	99016.86656
ADJUS*ED R SQUARE	.99986	RESIDUAL	93.	8.85127	.09517	
S*D DEVIA*ION	.30850	COEFF OF VARIABILI*Y	1.2 PC*			

*O*AL SUM OF SQUARES ADJUS*ED FOR MEAN OF DEPENDEN* VARIABLE
MUL*IPLE R .99169 R SQUARE .98344 ADJUS*ED R SQUARE .98237

—————— VARIABLES IN THE EQUA*ION —————— —————— VARIABLES NO* IN *HE EQUA*ION ——————

VARIABLE	B	STD ERROR B	F	BETA	VARIABLE	PARTIAL	TOLERANCE	F

			0	.06928
X7	1.2135929	.43868205E-01	765.32579	.0730129
			0	.06594
X3	1.6925923	.32806823E-01	2661.8059	.1404062
			0	.12968
X4	2.0543202	.34498877E-01	3545.8988	.1706003
			0	.15559

ALL VARIABLES ARE IN •HE EQUA•ION.

DEPENDENT VARIABLE.. Y ----- REGRESSION FORCED THROUGH ORIGIN ------

COEFFICIENTS AND CONFIDENCE INTERVALS.

VARIABLE	B	STD ERROR B	T	95.0 PCT CONFIDENCE INTERVAL		
X1	.99372973	.96879013E-02	102.57430	.97449148	.	1.0129680
X2	1.1174003	.27918481E-01	40.023678	1.0619597	.	1.1728409
X5	.35440880	.45260777E-01	7.8303738	.26452987	.	.44428773
X6	1.1146586	.34735451E-01	32.089941	1.0456808	.	1.1836363
X7	1.2135929	.43868205E-01	27.664522	1.1264794	.	1.3007065
X3	1.6925923	.32806823E-01	51.592692	1.6274445	.	1.7577402
X4	2.0543202	.34498877E-01	59.547450	1.9858122	.	2.1228281

VARIANCE/COVARIANCE MA•RIX OF •HE UNNORMALIZED REGRESSION COEFFICIEN•S.

X1	.00009						
X2	-.00006	.00078					
X3	-.00012	-.00011	.00108				
X4	-.00014	.00006	.00002	.00119			
X5	-.00007	-.00023	-.00012	-.00035	.00205		
X6	-.00009	-.00031	-.00006	-.00008	.00002	.00121	
X7	-.00018	.00008	-.00007	-.00013	-.00022	.00006	.00192
	X1	X2	X3	X4	X5	X6	X7

STEP	VARIABLE ENTERED	VARIABLE REMOVED	F TO ENTER OR REMOVE	SIGNIFICANCE	MULTIPLE R	R SQUARE	R SQUARE CHANGE	SIMPLE R	OVERALL F	SIGNIFICANCE
1	X1		10521.48772	0	.99001	.98011	.98011	.99001	99016.86656	0
	X2		1601.89484	0	.99239	.98584	.00572	.87431		
	X5		61.31475	0	.99359	.98722	.00139	.88083		
	X6		1029.76431	0	.99462	.98927	.00205	.88031		
	X7		765.32579	0	.99554	.99109	.00182	.89702		
	X3		2661.80588	0	.99737	.99475	.00366	.91964		
	X4		3545.89882	0	.99993	.99987	.00512	.92749		

1REGS1

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CPU TIME REQUIRED.. .633 SECONDS

FINISH

TOTAL CPU TIME USED.. .693 SECONDS

RUN COMPLETED

NUMBER OF CONTROL CARDS READ 10
NUMBER OF ERRORS DETECTED 0

S

14.48.01.UCLP, C2, X711P, [illegible] 0.256KLNS.

1	24.20	10	2	2	2	2	1	2
2	24.70	9	3	2	2	1	2	1
3	22.32	12	1	2	1	1	1	2
4	23.48	12	3	0	2	1	2	1
5	21.94	6	3	2	2	2	2	2
6	24.47	9	1	2	3	1	2	2
7	21.25	12	2	2	0	1	1	2
8	25.20	10	0	3	2	1	3	2
9	22.00	8	2	2	2	2	0	3
10	23.80	9	3	2	2	2	3	0
11	21.55	9	2	2	1	1	2	2
12	22.54	10	1	2	2	1	2	1
13	20.07	9	3	2	3	2	2	1
14	24.34	12	1	2	2	1	1	2
15	24.24	13	2	2	2	1	1	0
16	23.88	12	1	2	2	2	0	2
17	25.12	14	1	2	1	0	2	2
18	23.44	12	3	0	2	1	1	2
19	21.45	11	2	2	0	1	2	2
20	24.24	12	2	2	2	0	2	1
21	27.34	13	1	2	3	2	1	2
22	25.41	10	4	3	2	1	0	1
23	25.30	11	1	4	2	1	2	0
24	[illegible]	16	3	2	2	1	2	1
25	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

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1
S
                                                          86/12/20.    14.50.39.    PAGE    1

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   S P S S - - S*A*IS*ICAL PACKAGE FOR *HE SOCIAL SCIENCES

   VERSION 8.3 (NOS) — MAY 04, 1982

   376500 CM MAXIMUM FIELD LENGTH REQUEST


                 RUN NAME         REGS1
                 VARIABLE LIS*    A,Y,X1 *O X7
                 INPUT FORMAT     FREEFIELD
                 N OF CASES       100

CPU TIME REQUIRED..       .052 SECONDS


                 REGRESSION       VARIABLES=Y,X1 *O X7
                                  REGRESSION=Y WI*H X1 *O X7
                 OPTIONS          19
                 S*A*IS*ICS       ALL
                 READ INPUT DATA

            00053400 CM NEEDED FOR REGRESSION



            OPTION - 1
            IGNORE MISSING VALUE INDICATORS
            (NO MISSING VALUES DEFINED...OP*ION 1 MAY HAVE BEEN FORCED)

            OP*ION -19
            FORCE REGRESSION THRU THE ORIGIN


1REGS1                                                    86/12/20.    14.50.39.    PAGE    2

 FILE    NONAME    (CREA*ION DA*E =  86/12/20.)
0* * * * * * * * * * * * * * * * * * * * * * *   M U L T I P L E   R E G R E S S I O N   * * * * * * * * * * * * * * * * * * * * * * * * *
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A VALUE OF 99.00000 IS PRINTED
IF A COEFFICIENT CANNOT BE COMPUTED.

WARNING- COEFFICIENTS ARE NOT ADJUSTED FOR THE MEAN.

X1      -    .99311
X2           .92549    .90849
X3           .93893    .91479    .82786
X4           .92258    .91170    .83765    .80870
X5           .91795    .90521    .84644    .85658    .82948
X6           .92964    .90699    .86034    .89275    .82159    .84516
X7           .93887    .92748    .87569    .87731    .82881    .89516    .84811

               Y         X1        X2        X3        X4        X5        X6

1REGSI                                                       86/12/20.    14.50.39.    PAGE    3

 FILE    NONAME   (CREATION DATE =  86/12/20.)
0* * * * * * * * * * * * * * * * * * * * * * *  M U L T I P L E   R E G R E S S I O N  * * * * * * * * * * * * * * * * * * * * * *

0DEPENDENT VARIABLE..    Y                                         ----- REGRESSION FORCED THROUGH ORIGIN ------


 MEAN RESPONSE       27.90330      STD. DEV.      28.12263

 VARIABLE(S) ENTERED ON STEP NUMBER   1..   X1
                                            X5
                                            X6
                                            X4
                                            X2
                                            X3
                                            X7


 MULTIPLE R             .99988       ANALYSIS OF VARIANCE    DF      SUM OF SQUARES       MEAN SQUARE          F     SIGNIFICANC
E
 R SQUARE               .99977       REGRESSION               7.       79069.96202       11295.70886     57574.04967
0
 ADJUSTED R SQUARE      .99975       RESIDUAL                93.          18.24608            .19019
 STD DEVIATION          .44294       COEFF OF VARIABILITY     1.6 PCT

 TOTAL SUM OF SQUARES ADJUSTED FOR MEAN OF DEPENDENT VARIABLE
 MULTIPLE R  .99255          R SQUARE  .98515          ADJUSTED R SQUARE  .98419


 -------------------- VARIABLES IN THE EQUATION ---------------------         ---------- VARIABLES NOT IN THE EQUATION ----------
```

X4	1.8737566	.56846397E-01	1086.4768	.1290247
			0	.12020
X2	1.0910155	.46508167E-01	550.30426	.0237522
			0	.08753
X3	1.8183281	.50407039E-01	1301.2542	.1577157
			0	.14532
X7	1.1904395	.78255552E-01	231.41119	.0712106
			0	.06598

ALL VARIABLES ARE IN THE EQUATION.

0* M U L T I P L E R E G R E S S I O N *

DEPENDENT VARIABLE.. Y ----- REGRESSION FORCED THROUGH ORIGIN ------

COEFFICIENTS AND CONFIDENCE INTERVALS.

VARIABLE	B	STD ERROR B	T	95.0 PCT CONFIDENCE INTERVAL	
X1	1.0002352	.15711240E-01	63.663673	.96903581 ,	1.0314346
X5	.55589351	.70854192E-01	7.8455981	.41519113 ,	.69659589
X6	1.1152212	.54979294E-01	20.284386	1.0060432 ,	1.2243992
X4	1.8737566	.56846397E-01	32.961748	1.7608709 ,	1.9866423
X2	1.0910155	.46508167E-01	23.458580	.99865954 ,	1.1833716
X3	1.8183281	.50407039E-01	36.072901	1.7182297 ,	1.9184265
X7	1.1904395	.78255552E-01	15.212205	1.0350395 ,	1.3458395

VARIANCE/COVARIANCE MATRIX OF THE UNNORMALIZED REGRESSION COEFFICIENTS.

X1	.00025						
X2	-.00020	.00216					
X3	-.00027	.00034	.00254				
X4	-.00049	-.00017	.00037	.00323			
X5	-.00017	-.00016	-.00029	-.00031	.00502		
X6	-.00018	-.00060	-.00105	-.00001	-.00031	.00302	
X7	-.00042	-.00076	-.00065	.00051	-.00177	.00045	.00612
	X1	X2	X3	X4	X5	X6	X7

0* M U L T I P L E R E G R E S S I O N *

S U M M A R Y   T A B L E

STEP	VARIABLE ENTERED	VARIABLE REMOVED	F TO ENTER OR REMOVE	SIGNIFICANCE	MULTIPLE R	R SQUARE	R SQUARE CHANGE	SIMPLE R	OVERALL F	SIGNIFICANCE
1	X1		4053.06327	0	.99311	.98627	.98627	.99311	57574.04967	0
	X5		61.55341	0	.99411	.98826	.00200	.91795		
	X6		411.45632	0	.99612	.99225	.00399	.92964		
	X4		1086.47683	0	.99704	.99409	.00184	.92258		
	X2		550.30496	0	.99770	.99540	.00131	.92549		
	X3		1301.25418	0	.99960	.99920	.00379	.93893		
	X7		231.41119	0	.99988	.99977	.00057	.93887		

1REGS1 86/12/20. 14.50.39. PAGE 6

CPU TIME REQUIRED.. .649 SECONDS

FINISH

TOTAL CPU TIME USED.. .707 SECONDS

RUN COMPLETED

NUMBER OF CONTROL CARDS READ 10
NUMBER OF ERRORS DETECTED 0
S
14.52.12.UCLP, C2, XXXLP, 0.256KLNS.

1	22.52	8	3	1	2	2	3	1
2	20.84	10	1	2	1	2	1	2
3	21.42	9	2	1	2	3	2	1
4	20.12	9	0	2	1	2	2	2
5	19.72	11	1	0	2	1	1	2
6	[illegible]	7	3	2	3	2	2	1
7	24.34	12	1	2	2	1	1	2
8	24.24	13	2	2	2	1	1	0
9	23.38	12	1	2	2	2	0	2
10	25.19	14	1	2	1	0	2	2
11	23.44	12	3	0	2	1	1	2
12	21.46	11	2	2	0	1	2	2
13	24.94	12	2	2	2	0	2	1
14	27.34	13	1	2	3	2	1	2
15	30.10	16	3	2	2	1	2	1
16	30.43	16	2	2	2	0	3	2
17	[illegible]	12	2	1	3	2	3	1
18	24.74	16	0	2	3	1	3	1
19	27.40	13	2	2	3	2	0	1
20	25.14	[illegible]	5	0	3	1	3	2
21	27.31	12	3	3	2	1	0	2
22	27.16	13	2	1	3	2	1	2
23	30.10	16	2	2	2	0	3	1
24	[illegible]	[illegible]	1	2	2	1	1	1

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VERSION 8.3 (NOS) -- MAY 04, 1982

376500 CM MAXIMUM FIELD LENGTH REQUEST


                RUN NAME        REGS1
                VARIABLE LIS*   A,Y,X1 *O X7
                INPUT FORMAT    FREEFIELD
                N OF CASES      100

CPU TIME REQUIRED..      .053 SECONDS

                REGRESSION      VARIABLES=Y,X1 TO X7
                                REGRESSION=Y WITH X1  TO X7
                OP*IONS         19
                STATISTICS      ALL
                READ INPU* DA*A

        00053400 CM NEEDED FOR REGRESSION


        OP*ION - 1
        IGNORE MISSING VALUE INDICATORS
        (NO MISSING VALUES DEFINED...OPTION 1 MAY HAVE BEEN FORCED)

        OP*ION -19
        FORCE REGRESSION *HRU *HE ORIGIN
```



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FILE  NONAME  (CREA*ION DA*E = 86/12/20.)
* * * * * * * * * * * * * * * * * * * * * * * *   M U L * I P L E   R E G R E S S I O N   * * * * * * * * * * * * * * * * * * * * * * * * * *
```

VARIABLE	MEAN	S*ANDARD DEV	CASES
Y	29.4597	1.5311	100
X1	16.4280	1.6943	100
X2	1.[illegible]	.9773	100
[illegible]	1.[illegible]	[illegible]	100

```
A VALUE OF 99.00000 IS PRINTED
IF A COEFFICIENT CANNOT BE COMPUTED.

WARNING- COEFFICIENTS ARE NOT ADJUSTED FOR THE MEAN.

X1        .99360
X2        .88324    .86555
X3        .91335    .88959    .80056
X4        .92756    .90130    .77307    .79654
X5        .88567    .88894    .79660    .82803    .75518
X6        .90317    .89592    .81039    .76862    .82808    .78442
X7        .90761    .89090    .75341    .83571    .84967    .80180    .80568

          Y         X1        X2        X3        X4        X5        X6

IREGS1          .                                   86/12/20.    14.52.37.    PAGE    3

 FILE    NONAME    (CREATION DATE =  86/12/20. )
0* * * * * * * * * * * * * * * * * * * * * * *  M U L T I P L E   R E G R E S S I O N  * * * * * * * * * * * * * * * * * * * * * * *

0DEPENDENT VARIABLE..    Y                                        ----- REGRESSION FORCED THROUGH ORIGIN ------

 MEAN RESPONSE      29.46070      STD. DEV.      29.50537

 VARIABLE(S) ENTERED ON STEP NUMBER   1..   X1
                                            X2
                                            X5
                                            X7
                                            X6
                                            X3
                                            X4

 MULTIPLE R         .99994      ANALYSIS OF VARIANCE     DF      SUM OF SQUARES      MEAN SQUARE          F      SIGNIFICANC
F
 R SQUARE           .99988      REGRESSION               7.       87046.49556        12435.21379    113616.73394
0
 ADJUSTED R SQUARE  .99988      RESIDUAL                93.          10.17874            .10945
 STD DEVIATION      .33083      COEFF OF VARIABILITY    1.1 PCT

 TOTAL SUM OF SQUARES ADJUSTED FOR MEAN OF DEPENDENT VARIABLE
 MULTIPLE R  .9[illegible]      R SQUARE  .90135      ADJUSTED R SQUARE  .[illegible]

 -------------------- VARIABLES IN THE EQUATION --------------------      ---------- VARIABLES NOT IN THE EQUATION ----------
```

			0	.04417
X6	1.0192259	.51092252E-01	397.8897	.0521609
			0	.04705
X3	1.7562714	.35755511E-01	2412.6654	.1282182
			0	.11634
X4	2.0063207	.34141973E-01	3453.2140	.1624865
			0	.14214

ALL VARIABLES ARE IN THE EQUATION.

1REGS1 86/12/20. 14.52.37. PAGE 4

FILE NONAME (CREATION DATE = 86/12/20.)

0* M U L T I P L E R E G R E S S I O N *

0DEPENDENT VARIABLE.. Y ----- REGRESSION FORCED THROUGH ORIGIN ------

COEFFICIENTS AND CONFIDENCE INTERVALS.

VARIABLE	B	STD ERROR B	T	95.0 PCT CONFIDENCE INTERVAL		
X1	1.0081614	.90618423E-02	111.25347	.99016636	,	1.0261564
X2	1.0581480	.32444763E-01	32.613400	.99371909	,	1.1225768
X5	.30384955	.53004443E-01	5.8268615	.20359323	,	.41410587
X7	1.1804014	.63905282E-01	18.471109	1.0534982	,	1.3073047
X6	1.0192269	.51099952E-01	19.945751	.91775252	,	1.1207013
X3	1.7562714	.35755511E-01	49.118901	1.6852680	,	1.8272747
X4	2.0063207	.34141973E-01	58.764053	1.9335215	,	2.0741199

VARIANCE/COVARIANCE MATRIX OF THE UNNORMALIZED REGRESSION COEFFICIENTS.

X1	.00008						
X2	-.00007	.00105					
X3	-.00010	-.00019	.00128				
X4	-.00014	-.00001	.00003	.00117			
X5	-.00021	-.00018	-.00025	.00043	.00281		
X6	-.00019	-.00032	.00031	-.00015	.00009	.00261	
X7	-.00012	.00026	-.00052	-.00055	-.00029	-.00022	.00403
	X1	X2	X3	X4	X5	X6	X7

1REGS1 86/12/20. 14.52.37. PAGE 5

FILE NONAME (CREATION DATE = 86/12/20.)

0* M U L T I P L E R E G R E S S I O N *

0DEPENDENT VARIABLE.. Y ----- REGRESSION FORCED THROUGH ORIGIN ------

ENTERED	REMOVED		ENTER OR REMOVE				CHANGE			
1	X1		12377.33422	0	.99350	.98736	.98736	.99300	113616.73394	0
	X2		1063.66187	0	.99474	.98950	.00214	.88324		
	X5		33.95232	0	.99474	.98950	.00000	.88567		
	X7		341.18188	0	.99615	.99232	.00282	.90761		
	X6		397.83297	0	.99634	.99269	.00037	.90317		
	X3		2412.66639	0	.99777	.99554	.00285	.91335		
	X4		3453.21398	0	.99994	.99988	.00434	.92756		

TREGS1 86/12/20. 14.52.37. PAGE 6

CPU TIME REQUIRED.. .635 SECONDS

FINISH

TOTAL CPU TIME USED.. .694 SECONDS

RUN COMPLETED

NUMBER OF CONTROL CARDS READ 10
NUMBER OF ERRORS DETECTED 0
S
14.54.42.UCLP, C2, X711P, 0.256KLNS.

1	27.90	17	2	2	1	2	1	1
2	27.71	14	2	1	3	1	2	1
3	28.87	18	3	2	0	3	2	1
4	29.31	15	2	2	3	1	1	1
5	30.60	19	1	2	2	2	2	0
6	29.50	16	0	1	4	1	1	2
7	25.31	15	3	2	1	1	0	1
8	30.00	19	1	0	3	1	2	1
9	31.55	16	2	2	2	2	1	1
10	23.25	15	3	2	2	1	1	1
11	31.60	17	2	1	3	0	2	2
12	[illegible]	16	3	2	2	1	1	1
13	[illegible]	16	0	4	2	1	1	2
14	[illegible]	14	3	2	3	1	1	1
15	28.70	16	2	2	2	2	0	2
16	[illegible]	16	1	0	3	1	2	1
17	[illegible]	18	3	2	1	0	1	1
18	30.30	13	3	3	3	2	1	1
19	31.51	19	1	1	3	1	2	1
20	[illegible]	17	2	2	3	1	1	0
21	[illegible]	17	2	3	2	1	2	1
22	[illegible]	14	1	3	0	2	2	2
23	31.21	17	2	[illegible]	3	1	1	1
[illegible]	[illegible]	15	2	[illegible]	3	1	2	1
[illegible]	[illegible]	15	1	[illegible]	2	1	2	1
[illegible]	[illegible]	14	3	[illegible]	3	2	1	1
[illegible]	[illegible]	13	2	[illegible]	2	1	1	1
[illegible]	[illegible]	13	1	1	3	0	1	1
[illegible]	31.61	16	[illegible]	3	3	1	2	1
[illegible]	[illegible]	17	[illegible]	[illegible]	1	2	1	1

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                                                              86/12/20.    14.57.24.    PAGE   1

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   VERSION 8.3 (NOS) -- MAY 04, 1982

   376000 CM MAXIMUM FIELD LENGTH REQUEST

            JOB NAME        REGS1
            VARIABLE LIST   A,Y,X1 TO X6
            INPUT FORMAT    FREEFIELD
            N OF CASES      100

CPU TIME REQUIRED..       .051 SECONDS

            REGRESSION      VARIABLES=Y,X1 TO X6
                            REGRESSION=Y WITH X1 TO X6
            OPTIONS         19
            STATISTICS      ALL
            READ INPUT DATA

      00053400 CM NEEDED FOR REGRESSION

      OPTION - 1
      IGNORE MISSING VALUE INDICATORS
      (NO MISSING VALUES DEFINED...OPTION 1 MAY HAVE BEEN FORCED)

      OPTION -19
      FORCE REGRESSION THRU THE ORIGIN

REGS1                                                         86/12/20.    14.57.24.    PAGE   2

FILE    NONAME    (CREATION DATE = 86/12/20.)
* * * * * * * * * * * * * * * * * * * * * * * M U L T I P L E   R E G R E S S I O N * * * * * * * * * * * * * * * * * * * * * * *
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[illegible]	[illegible]	[illegible]	[illegible]
X3	1.5350	[illegible]	100
X4	2.2000	[illegible]	100
X5	1.2600	[illegible]	100
X6	1.3400	[illegible]	100

CORRELATION COEFFICIENTS.

A VALUE OF 99.00000 IS PRINTED
IF A COEFFICIENT CANNOT BE COMPUTED.

WARNING - COEFFICIENTS ARE NOT ADJUSTED FOR THE MEAN.

	Y	X1	X2	X3	X4	X5
X1	.99355					
X2	.89843	.87482				
X3	.91161	.88610	.82018			
X4	.93517	.90997	.79335	.80624		
X5	.88627	.88960	.79527	.82575	.77045	
X6	.90844	.89744	.82440	.77070	.84878	.77524

86/12/20. 14.57.24. PAGE 3

NREGS1

FILE NONAME (CREATION DATE = 86/12/20.) * M U L T I P L E R E G R E S S I O N *

------ REGRESSION FORCED THROUGH ORIGIN ------

DEPENDENT VARIABLE.. Y

MEAN RESPONSE 28.32700 STD. DEV. 28.36840

VARIABLE(S) ENTERED ON STEP NUMBER 1.. X1
X2
X5
X3
X6
X4

		ANALYSIS OF VARIANCE	DF	SUM OF SQUARES	MEAN SQUARE	F	SIGNIFICANC
MULTIPLE R	.99995						
R SQUARE	.99990	REGRESSION	6.	80468.54879	13411.42480	156252.06146	.00
ADJUSTED R SQUARE	.99989	RESIDUAL	94.	9.06921	.09584		
[illegible]	[illegible]	COEFF OF VARIABILITY	1.0 PCT				

VARIABLE	B	STD ERROR B	F	BETA
			SIGNIFICANCE	ELASTICITY
X1	.99324768	.78578931E-02	15977.281	.5740016
			.000	.57189
X2	1.1523991	.29316500E-01	1545.1890	.0876922
			.000	.07892
X5	.41649196	.47277830E-01	77.606513	.0207628
			.000	.01853
X3	1.6935843	.31389408E-01	2911.0349	.1267828
			.000	.11539
X6	1.0911604	.45369592E-01	578.42549	.0592145
			0	.05354
X4	2.0256152	.29985606E-01	4563.3965	.1743192
			0	.16161

----------- VARIABLES NOT IN THE EQUATION -----------

VARIABLE	PARTIAL	TOLERANCE	F
			SIGNIFICANCE

ALL VARIABLES ARE IN THE EQUATION.

FILE NONAME (CREATION DATE = 86/12/20.)

0* M U L T I P L E R E G R E S S I O N *

0DEPENDENT VARIABLE.. Y

----- REGRESSION FORCED THROUGH ORIGIN ------

COEFFICIENTS AND CONFIDENCE INTERVALS.

VARIABLE	B	STD ERROR B	T	95.0 PCT CONFIDENCE INTERVAL		
X1	.99324768	.78578931E-02	126.40127	.9775456	,	1.0089497
X2	1.1523991	.29316500E-01	39.308822	1.0941905	,	1.2105077
X5	.41649196	.47277830E-01	8.8094559	.32252072	,	.51036329
X3	1.6935843	.31389408E-01	53.954008	1.6312599	,	1.7559088
X6	1.0911604	.45369592E-01	24.050473	1.0010780	,	1.1812423
X4	2.0256152	.29985606E-01	67.552915	1.9650780	,	2.0851523

VARIANCE/COVARIANCE MATRIX OF THE UNNORMALIZED REGRESSION COEFFICIENTS.

X1	.00006			
X2	-.00005	.00086		
X3	-.00002	-.00013	.00099	
X4	-.00012	.00061	-.00004	.00090

DEPENDENT VARIABLE..   Y

----- REGRESSION FORCED THROUGH ORIGIN ------

S U M M A R Y   T A B L E

STEP	VARIABLE ENTERED	VARIABLE REMOVED	F TO ENTER OR REMOVE	SIGNIFICANCE	MULTIPLE R	R SQUARE	R SQUARE CHANGE	SIMPLE R	OVERALL F	SIGNIFICANCE
1	X1		15977.28138	.000	.99355	.98715	.98715	.99355	156252.06146	.000
	X2		1545.18897	.000	.99539	.99079	.00365	.89843		
	X5		77.60651	.000	.99539	.99079	.00000	.88627		
	X3		2911.03494	.000	.99685	.99371	.00291	.91181		
	X6		578.42549	0	.99751	.99503	.00133	.90844		
	X4		4563.39645	0	.99995	.99990	.00487	.93517		

1REGS1   86/12/20.   14.57.24.   PAGE 6

CPU TIME REQUIRED..   .583 SECONDS

FINISH

TOTAL CPU TIME USED..   .641 SECONDS

RUN COMPLETED

NUMBER OF CONTROL CARDS READ   10
NUMBER OF ERRORS DETECTED   0

S

14.59.43.UCLP, C2, X71LP   .   0.384KLNS.

1	29.15	18	0	2	3	1	1
2	27.64	15	2	2	2	2	2
3	25.35	19	1	0	2	1	1
4	27.25	16	2	2	2	1	1
5	27.35	17	1	1	3	1	1
6	27.40	16	1	3	2	1	1
7	27.80	18	3	3	0	1	1
8	27.50	13	3	2	3	1	1
9	26.50	15	2	0	3	2	2
10	29.50	15	2	2	3	1	2
11	27.37	16	1	3	2	1	1
12	22.00	13	4	2	3	0	2
13	23.05	13	2	3	3	2	1
14	25.45	18	2	2	0	2	1
15	27.14	16	0	3	2	2	1
16	26.00	16	1	3	3	1	[illegible]
17	27.34	15	2	1	3	2	[illegible]
18	25.44	18	2	2	0	2	1
19	27.73	17	3	0	3	1	1
20	[illegible]	[illegible]	2	1	2	[illegible]	[illegible]

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 COMPU*ER CEN*RE
UNIVERSITY OF BRADFORD

S P S S - - STATISTICAL PACKAGE FOR *HE SOCIAL SCIENCES

VERSION 8.3 (NOS) -- MAY 04, 1982

376500 CM MAXIMUM FIELD LENG*H REQUES*


                RUN NAME        REGS1
                VARIABLE LIST   A,Y,X1 TO X5
                INPU* FORMA*    FREEFIELD
                N OF CASES      100

CPU *IME REQUIRED..       .049 SECONDS


                REGRESSION      VARIABLES=Y,X1 TO X5
                                REGRESSION=Y WITH X1 TO X5
                OP*IONS         19
                STATISTICS      ALL
                READ INPU* DA*A

          00053400 CM NEEDED FOR REGRESSION



          OP*ION - 1
          IGNORE MISSING VALUE INDICATORS
          (NO MISSING VALUES DEFINED...OPTION 1 MAY HAVE BEEN FORCED)

          OPTION -19
          FORCE REGRESSION *HRU *HE ORIGIN
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FILE   NONAME   (CREA*ION DA*E = 86/12/20. )
0* * * * * * * * * * * * * * * * * * * * * * * * *  M U L * I P L E   R E G R E S S I O N  * * * * * * * * * * * * * * * * * * * * * * *


VARIABLE          MEAN       S*ANDARD DEV       CASES

Y                20.0114         1.3679          100
X1               10.8300         1.3044          100
```

WARNING- COEFFICIENTS ARE NOT ADJUSTED FOR THE MEAN.

X1	.99340				
X2	.91309	.88380			
X3	.92481	.90050	.81950		
X4	.92300	.89150	.85951	.82701	
X5	.88154	.86969	.83334	.84240	.76486
	Y	X1	X2	X3	X4

FILE NONAME (CREATION DATE = 86/12/20.)

0* M U L T I P L E R E G R E S S I O N *

0DEPENDENT VARIABLE.. Y ----- REGRESSION FORCED THROUGH ORIGIN ------

MEAN RESPONSE 26.61180 STD. DEV. 26.67662

VARIABLE(S) ENTERED ON STEP NUMBER 1.. X1
X5
X4
X2
X3

MULTIPLE R	.99980	ANALYSIS OF VARIANCE	DF	SUM OF SQUARES	MEAN SQUARE	F	SIGNIFICANC
R SQUARE	.99959	REGRESSION	5.	71135.31608	14227.06322	46800.00940	
ADJUSTED R SQUARE	.99958	RESIDUAL	96.	28.87972	.30400		
STD DEVIATION	.55136	COEFF OF VARIABILITY	2.1 PCT				

TOTAL SUM OF SQUARES ADJUSTED FOR MEAN OF DEPENDENT VARIABLE
MULTIPLE R .95723 R SQUARE .91639 ADJUSTED R SQUARE .91287

---------------------- VARIABLES IN THE EQUATION ---------------------- ---------- VARIABLES NOT IN THE EQUATION ----------

VARIABLE	B	STD ERROR B	F / SIGNIFICANCE	BETA / ELASTICITY	VARIABLE	PARTIAL	TOLERANCE	F / SIGNIFICANCE

ALL VARIABLES ARE IN THE EQUATION.

COEFFICIENTS AND CONFIDENCE INTERVALS.

VARIABLE	B	STD ERROR B	T	95.0 PCT CONFIDENCE INTERVAL	
X1	1.0316865	.10690969E-01	96.446620	1.0104503	, 1.0529227
X5	.29630887	.59142424E-01	5.0100901	.17889632	, .41372143
X4	1.9505563	.69548671E-01	28.045917	1.8124847	, 2.0886278
X2	1.0232365	.53327226E-01	19.187882	.91736859	, 1.1291044
X3	1.6278417	.66055300E-01	24.643619	1.4967053	, 1.7589780

FILE NONAME (CREATION DATE = 86/12/20.)
0* M U L T I P L E R E G R E S S I O N *

0DEPENDENT VARIABLE.. Y ----- REGRESSION FORCED THROUGH ORIGIN ------

VARIANCE/COVARIANCE MATRIX OF THE UNNORMALIZED REGRESSION COEFFICIENTS.

X1	.00011				
X2	-.00013	.00284			
X3	-.00030	.00004	.00436		
X4	-.00032	-.00135	-.00062	.00484	
X5	-.00021	-.00096	-.00107	.00077	.00350
	X1	X2	X3	X4	X5

FILE NONAME (CREATION DATE = 86/12/20.)
0* M U L T I P L E R E G R E S S I O N *

0DEPENDENT VARIABLE.. Y ----- REGRESSION FORCED THROUGH ORIGIN ------

S U M M A R Y T A B L E

STEP	VARIABLE	F TO	SIGNIFICANCE	MULTIPLE R	R SQUARE	R SQUARE	SIMPLE R	OVERALL F	SIGNIFICANCE
ENTERED	REMOVED	ENTER OR REMOVE				CHANGE			
1	X1	[illegible]	0	[illegible]	.99695	[illegible]	[illegible]	[illegible]	0

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                                                                        86/12/20.    15.03.19.     PAGE    1

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 S P S S - - STATISTICAL PACKAGE FOR THE SOCIAL SCIENCES

 VERSION 8.3 (NOS) -- MAY 04, 1982

 376500 CM MAXIMUM FIELD LENGTH REQUEST


           RUN NAME        REGS1
           VARIABLE LIS*   A,Y,X1 *O X7
           INPUT FORMA*    FREEFIELD
           N OF CASES      100

CPU *IME REQUIRED..      .053 SECONDS


           REGRESSION      VARIABLES=Y,X1 TO X7
                           REGRESSION=Y WITH X1 TO X7
           OP*IONS         19
           STATISTICS      ALL
           READ INPU* DA*A

       00053400 CM NEEDED FOR REGRESSION


       OP*ION - 1
       IGNORE MISSING VALUE INDICA*ORS
       (NO MISSING VALUES DEFINED...OPTION 1 MAY HAVE BEEN FORCED)

       OPTION -19
       FORCE REGRESSION *HRU *HE ORIGIN


REGS1                                                                   86/12/20.    15.03.19.     PAGE    2

FILE    NONAME   (CREATION DATE = 86/12/20.)
 * * * * * * * * * * * * * * * * * * * * * * *   M U L * I P L E   R E G R E S S I O N   * * * * * * * * * * * * * * * * * * * * * * *
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WARNING- COEFFICIEN*S ARE NO* ADJUS*ED FOR *HE MEAN.

	Y	X1	X2	X3	X4	X5	X6
X1	.95904						
X2	.87200	.83947					
X3	.90700	.87963	.77935				
X4	.91655	.88185	.82101	.75679			
X5	.90310	.89712	.77094	.82890	.78739		
X6	.92597	.83823	.86418	.84258	.85154	.83852	
X7	.91195	.90469	.72100	.79908	.80865	.83742	.80953

FILE NONAME (CREATION DATE = 86/12/20.)

0* M U L T I P L E R E G R E S S I O N *

0DEPENDEN* VARIABLE.. Y ----- REGRESSION FORCED *HROUGH ORIGIN ------

MEAN RESPONSE 24.52360 S*D. DEV. 24.58635

VARIABLE(S) EN*ERED ON S*EP NUMBER 1.. X1
X2
X3
X4
X5
X7
X6

		ANALYSIS OF VARIANCE	DF	SUM OF SQUARES	MEAN SQUARE	F	SIGNIFICANCE
MUL*IPLE R	.99994						
R SQUARE	.99989	REGRESSION	7.	60442.05757	8634.57965	118302.24974	
ADJUSTED R SQUARE	.99988	RESIDUAL	93.	6.78783	.07299		
STD DEVIATION	.27016	COEFF OF VARIABILITY	1.1 PCT				

TOTAL SUM OF SQUARES ADJUSTED FOR MEAN OF DEPENDENT VARIABLE
MULTIPLE R .98892 R SQUARE .97797 ADJUSTED R SQUARE .97655

------------------ VARIABLES IN *HE EQUA*ION ------------------ ---------- VARIABLES NOT IN THE EQUATION ----------

VARIABLE	B	S*D ERROR B	F / SIGNIFICANCE	BE*A / ELAS*ICI*Y	VARIABLE	PAR*IAL	*OLERANCE	F / SIGNIFICANCE

```
X7        1.2305447      .34699363E-01   1257.6457        .0948311
                                                 0         .08580
X6        1.0382742      .36123485E-01    826.12319       .0842480
                                                 0         .07799
```

ALL VARIABLES ARE IN THE EQUATION.

FILE NONAME (CREATION DATE = 86/12/20.)

0* M U L T I P L E R E G R E S S I O N *

0DEPENDENT VARIABLE.. Y

----- REGRESSION FORCED THROUGH ORIGIN ------

COEFFICIENTS AND CONFIDENCE INTERVALS.

VARIABLE	B	STD ERROR B	T	95.0 PCT CONFIDENCE INTERVAL	
X1	1.0135976	.89112120E-02	113.74408	.99520172 ,	1.0312935
X2	1.2081203	.35342823E-01	34.182913	1.1379370 ,	1.2783047
X3	1.6781572	.30792706E-01	54.498530	1.6170090 ,	1.7393054
X4	1.9903533	.34025842E-01	58.495343	1.9227847 ,	2.0579219
X5	.32266818	.45850391E-01	7.0374140	.23161839 ,	.41371797
X7	1.2305447	.34699363E-01	35.463033	1.1616386 ,	1.2994507
X6	1.0382742	.36123485E-01	28.742359	.96654008 ,	1.1100082

VARIANCE/COVARIANCE MATRIX OF THE UNNORMALIZED REGRESSION COEFFICIENTS.

X1	.00008						
X2	-.00006	.00125					
X3	-.00009	-.00008	.00095				
X4	-.00011	-.00027	.00020	.00116			
X5	-.00014	-.00002	-.00018	.00011	.00210		
X6	-.00002	-.00049	-.00027	-.00029	-.00026	.00130	
X7	-.00015	.00027	-.00002	-.00012	-.00022	-.00009	.00120
	X1	X2	X3	X4	X5	X6	X7

FILE NONAME (CREATION DATE = 86/12/20.)

0* M U L T I P L E R E G R E S S I O N *

0DEPENDENT VARIABLE.. Y

----- REGRESSION FORCED THROUGH ORIGIN ------

S U M M A R Y T A B L E

STEP	VARIABLE ENTERED	VARIABLE REMOVED	F TO ENTER OR REMOVE	SIGNIFICANCE	MULTIPLE R	R SQUARE	R SQUARE CHANGE	SIMPLE R	OVERALL F	SIGNIFICANCE
1	X1		12937.71616	0	.98904	.97821	.97821	.98904	118302.24974	0
	X2		1168.47191	0	.99310	.98625	.00804	.87900		
	X3		2970.08979	0	.99525	.99052	.00427	.90700		
	X4		3421.70519	0	.99841	.99682	.00630	.91655		
	X5		49.52520	0	.99859	.99719	.00037	.90316		
	X7		1257.62671	0	.99944	.99889	.00170	.91195		
	X6		826.12319	0	.99994	.99989	.00100	.92599		

CPU TIME REQUIRED.. .662 SECONDS

FINISH

TOTAL CPU TIME USED.. .721 SECONDS

RUN COMPLETED

NUMBER OF CONTROL CARDS READ 10
NUMBER OF ERRORS DETECTED 0

S
15.04.28.UCLP, C2, XLLP 0.256KLNS.

1	23.05	12	0	2	1	1	2	3
2	21.50	9	2	0	2	1	3	2
3	22.93	13	1	2	0	2	1	3
4	22.40	10	3	2	1	0	2	1
5	23.05	14	2	1	2	1	0	1
6	22.00	11	1	2	2	1	2	0
7	25.75	12	2	1	3	1	2	1
8	24.70	14	1	2	1	1	1	2
9	25.50	13	1	2	2	1	2	1
10	24.13	10	2	2	1	2	2	3
11	23.05	9	3	2	3	2	3	2
12	24.00	11	2	1	2	2	1	1
13	21.60	11	2	2	1	2	2	0
14	24.20	12	1	3	1	1	0	3
15	27.05	9	3	3	2	0	2	2
16	22.55	14	1	2	0	1	1	2
17	23.05	12	2	0	2	1	2	2
18	20.33	11	0	3	2	2	2	3
19	21.40	14	1	0	1	1	1	2
20	21.55	11	3	2	0	1	2	1
21	24.55	13	1	1	2	0	2	2
22	21.25	10	2	2	2	1	0	1
23	23.55	10	2	1	3	1	3	0
24	24.70	13	1	2	1	1	2	2
[illegible]	[illegible]	12	2	1	2	1	2	[illegible]